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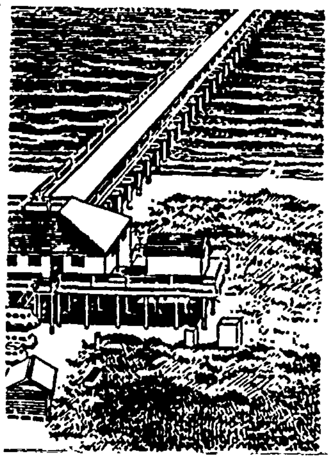

MONITORING COMPLETED COASTAL PROJECTS PROGRAM

MISCELLANEOUS PAPER CERC-91-10

MONITORING OF COMPLETED BREAKWATERS AT CATTARAUGUS CREEK HARBOR, NEW YORK

by

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Under the Monitoring Completed Coastal Projects Program, an assessment of shore-connected breakwaters and channel improvements at Cattaraugus Creek Harbor, New York, was performed. Data collection for the monitoring effort occurred from mid-1983 through 1985. An evaluation of waves, structure stability, sediment transport, channel stability, and ice-jam problems due to the construction of the project is discussed in detail. Data on the performance of the project and observations made in the prototype, as well as conclusions and recommendations based on the monitoring effort, are reported herein.

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See reverse.

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Breakwaters
Cattaraugus Creek Harbor, New York
Channel stability
Ice-jam problems
Sediment transport
Structure stability
Waves

PREFACE

Funding for the study report herein was provided through the Monitoring Completed Coastal Projects (MCCP) Program. The program entails intense monitoring of selected Civil Works coastal projects to assure adequate information as a basis for improving project purpose attainment, design procedures, construction methods, and operation and maintenance techniques. Overall program management is by the Hydraulic Design Section of Headquarters, US Army Corps of Engineers (HQUSACE). The Coastal Engineering Research Center (CERC), US Army Engineer Waterways Experiment Station (WES), is responsible for technical and data management and support for HQUSACE review and technology transfer. Technical Monitors for the MCCP Program are Messrs. John H. Lockhart, Jr.; John G. Housley; James E. Crews; and Robert H. Campbell. The Program Manager is Ms. Carolyn M. Holmes, CERC.

This report was prepared by Messrs. Michael C. Mohr, US Army Engineer District, Buffalo (CENCB), J. Michael Hemsley, former Program Manager of MCCP and Robert R. Bottin, Jr., Wave Processes Branch, Wave Dynamics Division under the general supervision of Mr. Charles C. Calhoun, Jr., and Dr. James R. Houston, Assistant Chief and Chief of CERC, respectively.

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Commander and Director of WES during the report publication was COL Larry B. Fulton, EN. Technical Director was Dr. Robert W. Whalin.

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CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet per second	0.02831685	cubic metres per second
cubic yards	0.7646	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
inches	2.54	centimetres
miles (US statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms
square miles (US statute)	2.589988	square kilometres
tons (2,000 pounds, mass)	907.1847	kilograms

MONITORING OF COMPLETED BREAKWATERS AT CATTARAUGUS CREEK HARBOR, NEW YORK

PART I: INTRODUCTION

Geographical Setting and Project History

1. The Cattaraugus Creek Harbor improvements (channel improvements and shore-connected breakwaters) were completed in January 1983, at the mouth of Cattaraugus Creek in Lake Erie (Figure 1). Improvements were implemented at the creek mouth and extended along the creek center line 1 mile* upstream from the mouth. Cattaraugus Creek enters Lake Erie approximately 24 miles southwest of Buffalo, NY, and 54 miles northeast of Erie, PA, as shown on Figure 2. The town of Hanover in Chautauqua County is on the southern side of the harbor, and the town of Brant, Erie County, is on the north of the harbor. The Cattaraugus Indian Reservation of the Seneca Nation, New York Indians, occupies the entire northern side of the creek within the project area. The economy of the area is primarily recreational, and most of the residences are summer cottages. Cattaraugus Creek attracts patrons from well beyond the limits of local communities because of its location near good recreational fishing areas in Lake Erie and the scarcity of similar facilities to meet the increasing demands of small boat owners. Most residential and boat owners reside in the Buffalo area. Recreational fishermen come from a large area, as far away as the Midwest, as a result of State programs that stock Coho and Chinook salmon and rainbow trout.

2. The Cattaraugus Creek watershed covers an area of 558 square miles in western New York as shown on Figure 3. Cattaraugus Creek is about 70 miles long and flows in a general westward direction to Lake Erie. Terrain in the basin varies from the hilly, steep-sloped and narrow-valleyed portion of the basin upstream of Gowanda to the flat-sloped and wide-valleyed Lake Erie plain downstream of Gowanda. The regional climate is temperate and humid-continental and is characterized by rapidly changing weather. The prevailing winds at the project site are southwesterly from over Lake Erie. Strong winds

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

are common throughout the year because of the exposure. Waves incident to the site predominately have a high steepness with a very minor presence of swell due to the lake's size being the same order as that of the atmospheric disturbances. Fall deepwater wave heights of 10.2, 10.8, and 11.5 ft with periods of 8.3, 8.6, and 8.9 sec have a recurrence interval of 5-, 10- and 20-years, respectively. In addition to wave activity, water levels at the site vary yearly in response to long-term fluctuations in the components of the hydrologic cycle, seasonally due to relative changes in runoff and evaporation, and daily in response to storms over the lake. Tidal fluctuations are negligible. Average monthly levels have fluctuated 5.01 ft over the period of record from 1900-1988. Seasonal fluctuations in the average monthly level vary approximately 1.2 ft with the highest seasonal level occurring in June and the lowest in February. Short-term fluctuations due to wind setup can be quite large because the orientation of Lake Erie's long axis parallels major storm tracks and the lake's shallowness (average depth 62 ft). A wind setup of approximately 7.6 ft at Buffalo, NY, was recorded during the 2 December 1985 storm, the largest on record since 1900.

3. Two climatological stations are located within the watershed, one at Gowanda State Hospital (42°29′ N and 78°56′ E) and the other at Arcade (42°32′ N and 78°25′ E) (Figure 3). Streamflow data from the US Geological Survey (USGS) gage at Gowanda was used during project design. The drainage area for the Gowanda gage is 436 square miles, compared with the 558 square miles at the river mouth. Discharges for design floods were increased proportionally by drainage area to account for the differences in watershed areas between the gage and river mouth.

4. Prior to construction of the Cattaraugus Harbor improvements during the 1982 construction season, flooding occurred almost every year along the lower reaches of the creek. Although each of the past recorded flood events was coincident with a high discharge in the creek, most of the higher stages were aggravated by ice jams near the mouth. Sandbars frequently formed across the mouth of the creek (Figures 4 and 5). In the late winter and early spring, lake ice is windrowed onshore by lake storms, increasing the barrier effect of the bars. These obstacles impede access of creek discharges to the lake and, therefore, affect water level stages near the mouth. The effects of these obstructions are particularly apparent during the spring thaw when ice flows carried down the creek by the high discharges are blocked by the sandbar and lake ice at the creek mouth. During discharges developed by the

combination of rainfall and snowmelt, ice jams raise the water surface to significantly higher levels than would occur in an open channel. Navigation difficulties were also experienced at the mouth of the creek due to the shallow depths and the constant shifting of the bar across the entrance. Rectification of these problems led to the construction of the harbor improvements. Although the Cattaraugus Creek project is a multipurpose project that is expected to provide for some flood control, it is primarily a small-boat navigation project.

Previous Studies

Hydraulic model study

5. During 1975, a model study (Bottin and Chatham 1975) was conducted at the US Army Engineer Waterways Experiment Station (WES) to test the improvement plans proposed in the Cattaraugus Creek Harbor Phase I General Design Memorandum (US Army Engineer District (USAED), Buffalo 1975). The purposes of the study were to:

- a. Study shoaling, wave action, current patterns, and flood and ice flow conditions at the harbor entrance and lower reaches of the creek with the proposed improvements and revisions installed in the model.
- b. Develop remedial plans to alleviate undesirable conditions as found necessary.
- c. Determine whether suitable design modifications of the proposed plans could be made that would reduce construction costs significantly and still provide adequate protection.

6. The Cattaraugus Creek hydraulic model was constructed to a linear scale of 1:75, model to prototype. It reproduced the lower 5,400 ft of creek channel and underwater contours in Lake Erie to offshore depths ranging from -38 to -18 ft Low Water Datum (LWD).* The total area reproduced in the model represented about 3.3 square miles in the prototype (Figure 6). Nine improvement plans were tested, each including breakwaters in Lake Erie at the mouth of Cattaraugus Creek, totaling about 2,300 ft in length; a berm, about 700 ft in length and extending northward from the inner end of the north breakwater to high ground; a 120-ft-wide, 8-ft-deep entrance channel, about 1,900 ft in length and extending from the -8 ft LWD contour in the lake to the maneuvering

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* LWD = 568.6 ft above mean water level at Father Point, Quebec (International Great Lakes Datum (IGLD) 1955).

area; a 6-ft-deep maneuvering area 300 ft wide by 600 ft long; a 120-ft-wide, 6-ft-deep channel, extending upstream approximately 1,900 ft from the maneuvering area; a riprapped friction section extending about 750 ft upstream from the navigation channel through the New York Central Railroad bridge; two levees on the left bank totaling about 770 ft in length; and development of recreational facilities. A 60-ft-long wave generator, granulated nylon and crushed coal tracer materials, a model circulation system to reproduce streamflow, ice simulation materials, and an automated data acquisition and control system were used in the model operation. Plan 8 (Figure 7) most closely represented the plan that was constructed. The notable differences are that initial channel bottoms are -5.5 and -3.5 ft LWD instead of -8 and -6 ft LWD, and the maneuvering area was moved farther downstream to approximately 1,200 ft from the -8 ft LWD contour. The friction section and levees were eliminated, and crest height in the model was +10 ft LWD instead of the +12.5 ft LWD crest used during final design. These changes occurred as a result of the model study and better information during final design.

7. Tests to determine wave heights, wave-induced current patterns and magnitudes, sediment (tracer) movements, wave patterns, water-surface elevations, creek current velocities, and ice flows were conducted for base (no breakwaters) conditions and for the nine variations in the design elements of two basic harbor configurations. Wave height tests for the base conditions and Plan 8 were conducted for three directions (northwest, west, and west-southwest) with wave heights presented in Table 1 for two of those directions. These tests were conducted with water levels of +3.0 and +6.8 ft LWD. For the +3.0 ft still-water level (SWL), wave heights at the creek mouth for base conditions ranged from 1.0 to 1.8 ft, and 0.1 to 0.3 ft in the lower reaches of the creek, compared with 0.2 to 0.3 ft at the mouth and 0.1 ft or less for lower reaches of the creek for Plan 8. With a water level of +6.8 ft LWD, wave heights ranged from 3.3 to 5.9 ft at the creek mouth and from 0.2 to 1.2 ft in the lower reaches of the creek for base conditions, and 0.4 to 1.2 ft at the creek mouth, and did not exceed 0.1 ft in the lower reaches for Plan 8. Figure 8 shows wave patterns resulting from 9 ft, 6 sec waves from the northwest.

8. Wave-induced current patterns and magnitudes were determined at selected locations by timing the progress of a dye tracer relative to a known distance on the model surface using the same test directions and waves as for the wave height tests. Velocities ranged from 1.4 to 3.8 fps along the

Table 1

Modeled Wave Heights

Deepwater Wave Direction	Selected Shallow-Water Test Direction	Selected Test Waves Period sec	Selected Test Waves Height ft
Northwest	N 40° 00′ W	6	5
		6	9
West	N 79° 00′ W	6	7
		6	14
		9	7
		9	14

shoreline north of the creek mouth, from 0.8 to 1.9 fps inside the creek mouth, and from 1.1 to 2.7 fps along the shoreline south of the creek mouth for base conditions. For Plan 8, velocities ranged from 1.3 to 2.9 fps along the shoreline north of the north breakwater, from 1.0 to 4.3 fps in the entrance channel, and from 0.6 to 3.2 fps along the shoreline south of the south breakwater.

9. Tracer tests were conducted for test waves from northwest, west, and west-southwest, and spits were formed across the creek mouth for base conditions. While under wave attack, the spits were subjected to various creek discharges that shifted the shoaling patterns lakeward. Waves from northwest deflected creek currents and sediment from the eroding spit in a southerly direction as they entered the lake, while waves from west and west-southwest deflected creek currents and sediment from the eroding spit to the north as they entered the lake. With Plan 8 installed, waves from northwest moved some tracer across the entrance, and a small deposit occurred at the head of the south breakwater. Waves from west and west-southwest caused no deposits in the entrance channel. Figure 9 shows tracer deposits for 9-ft, 6-sec waves from the northwest for Plan 8.

10. Because ice jams are a major cause of flooding of the area near the mouth, qualitative tests were conducted to determine any ice-jamming tendencies between the breakwater structures. A low-density polyethylene sheet material, recommended by the US Army Engineer Cold Regions Research and Engineering Laboratory (CRREL), was used to simulate ice. This material, with a density similar to that of ice, was cut to represent ice fragments ranging from 5 to 25 feet square with a 1.5-ft prototype thickness. The required

sizes were estimated from photographs taken of previous ice conditions in the area. For the base test, the ice material was placed in an area across the creek mouth and upstream several hundred feet (after a spit was formed by waves from westsouthwest) and subjected to creek discharges of 5,000 and 10,000 cfs in an effort to simulate an ice jam for base conditions (Figures 10 and 11). Ice flow tests conducted for Plan 8 revealed no ice-jamming tendencies between the breakwaters for various creek discharges (Figure 12). The tests conducted with wave action from northwest revealed that the ice material was held between the breakwaters for a 1,000-cfs discharge and moved out of the channel and around the south breakwater for discharges of 5,000 cfs or greater. An ice jam constructed between the breakwaters (Figure 13) to simulate windrowed lake ice required a 10,000-cfs discharge to break the jam loose.

11. Creek current velocity measurements and water-surface profiles were secured along the channel center line for discharges of 10,000 to 50,000 cfs (in multiples of 10,000 cfs); and 57,000 cfs with a lake level of 0.0-ft LWD; and 40,000 and 57,900 cfs with a +3.0-ft LWD lake level. Visual observations revealed overbank flooding for creek discharges of 40,000 cfs and above for both lake levels for base conditions. Velocities at the creek mouth ranged from 7.7 fps for the 10,000-cfs discharge to 23.1 fps for the 57,900-cfs discharge with the 0.0-ft LWD lake level. Velocities between the breakwaters ranged from 6.3 fps for the 10,000-cfs discharge to 17.3 fps for the 50,000- and 57,900-cfs discharges at a lake level of 0.0-ft LWD, and 15.4 fps between the breakwaters at the +3.0-ft LWD lake level. Absolute values of current velocities for discharges of 40,000 cfs and above may be suspect since overbank roughness was not faithfully reproduced in the model.

Littoral and fluvial <u>sediment transport studies</u>

12. The General Design Memorandum studies for Cattaraugus Creek Harbor included a qualitative evaluation of sediment transport patterns for both fluvial and littoral processes (USAED, Buffalo 1976). In addition, quantitative predictions were made of the proposed project's impact on these processes. These studies included estimates of the creek bed load and the identification of a tendency for shoaling in the proposed dredged channel.

13. The project site is located on a shore trending approximately north-northeastward and is exposed to a wide sector of westerly waves. Cattaraugus Creek enters the lake in the center of a 3-mile-long, gently

arcuate embayment. The embayment has a sand and gravel beach extending from Hanover Bay in the south to Lotus Bay at the north end. Cattaraugus Creek is believed to be the primary source of littoral drift for the embayment. Other sources of littoral drift include Silver and Walnut Creeks, material eroded from the bluffs to the southwest of Hanover Bay, and material from more remote beaches. The net direction of littoral drift in the area is from south to north. The direction of littoral drift was determined from lithological studies of the nearshore sediment composition. The net littoral drift rate was estimated by wave-energy budget calculations to be approximately 40,000 cubic yards/year toward the north. Major storm waves from the westerly and southwesterly directions transport littoral drift northward. Reversals in the drift patterns from north and northeast are considerably shorter in duration than those from the southwesterly directions. The developments behind beaches to the south of the project site have been threatened by beach erosion damage caused by high lake levels; however, the long-term net drift appears to be in balance with the sediment supply.

14. Based upon the littoral transport analysis, the potential effect of the harbor structures on adjacent shorelines was estimated. It was assumed that the south breakwater would act as an impermeable groin impounding littoral drift on the south beach and effectively cutting off the littoral drift supply to the downdrift beach to the northeast. Prior to construction of the breakwaters, littoral drift was supplied to the northeast beach from Cattaraugus Creek bed load and material that passed the creek mouth from sources to the south. Construction of the breakwaters would reduce this supply, and dredging of the channel would reduce the quantity of material entering the beach system from the creek. These factors would cause a net accretion and erosion of the beaches to the south and north, respectively. The breakwaters would also reduce the effect of Cattaraugus Creek as a source for the southerly beaches during reversals in littoral transport to the south. By the application of the Pelnard-Conside're method (USAED, Buffalo 1976), which predicts the configuration of the fillets impounded or eroded at a groin given the transport rate and angle of predominant wave attack, a predicted shoreline map was developed (Figure 14).

15. Estimates for mean annual sediment transport at the mouth of Cattaraugus Creek were made using meander migration rates, suspended sediment concentration, sediment yield, bed-load transport equations, and computer simulation. The lateral movement of the stream channel provided a minimum

n asure of the bed material transport rate by assuming that most of the coarse sediment removed from a cut bank deposited on the point bar immediately downstream. Measurements of the rates of bank retreat were taken from aerial photographs taken in 1938, 1958, 1964, and 1971, with the height of the banks and composition determined through field studies. This method resulted in an estimation of 12,000 tons per year for the bed-load transport rate.

16. The total sediment load was estimated from the suspended sediment rating curve at Gowanda, NY, and flow-duration curve increased by drainage area to the mouth. Assuming a 10-percent increase for bed load based on Archer and LaSala's (1968) work in neighboring watersheds, average annual bed load and total sediment rates of 71,000 and 780,000 tons, respectively, were obtained.

17. Another estimate of the total load supplied by the drainage basin was made by the US Soil Conservation Service (SCS). Considering the erodability of the soils, land usage, and percentage of the area in each category, the SCS estimated the total yearly sediment load of Cattaraugus Creek to be about 520,000 tons.

18. The bed load was also determined using equations developed by Einstein (1950), Meyer-Peter and Muller (1948), and Kalinskee (1947). To represent the transport rate of each size component of the bed, the total sample was subdivided into five fractions, each containing 20 percent of the material. The minimum size considered was 1.0 mm. Potential load was computed by all three methods assuming the entire bed to consist of one size range. Total bed load for any given discharge was computed by summation of the transport ratio for each size range multiplied by 0.2. Total average annual bed load was determined by coupling the bed-load-discharge curve with discharge-duration information. This resulted in estimated average annual bed-load values of 12,000, 10,000, and 40,000 tons per year using the Einstein, Meyer-Peter and Muller, and Kalinskee equations, respectively. These values exclude the washload and that portion of the bed material considered to move in suspension (diameter less than 1.0 mm).

19. Potential rates of deposition in the entrance channel were determined using the computer program, "Scour and Deposition in Rivers" (US Army Engineer Hydrologic Engineering Center 1974). Using measured discharges for the years 1971 and 1972, hydraulic characteristics of the creek, Einstein's bed-load equation, and varying lake levels, the bed load and potential aggradation/degration of the channel were computed. The 2-year sequence,

1971-1972, included an exceptionally dry year and a wet year. The average annual bed-load transport rate was considered the average of the transport rates for the 2 years and approximated 35,000 tons per year. This value included material sizes ranging from fine sand and larger. Average deposition rates in the channel of 0.1 to 0.2 ft per year were also determined. Table 2 summarizes the average annual sediment transport values determined by the aforementioned methods.

Table 2

Estimates of Average Annual Sediment Transport at the Mouth of Cattaraugus Creek

Method	Total Load tons/year	Bed Load tons/year
Meander migration rates		12,000
Suspended sediment concentration	780,000	780,000
Sediment yield	520,000	--
Bed-load transport equation:		
Einstein		12,000
Meyer-Peter and Muller		10,000
Kalinskee		40,000
Computer simulation		35,000

Implemented Harbor Improvements

20. The improvement of Cattaraugus Creek Harbor was authorized for construction by Public Law 90-483, approved 13 August 1968, as presented in House Document No. 97, 90th Congress, 1st Session, dated 5 April 1967. The project (Figure 15) consists of two breakwaters in Lake Erie at the mouth of Cattaraugus Creek with personnel guardrails and 4-ft-wide walkways; a north breakwater 600 ft long with a berm 550 ft long and south breakwater 1,850 ft long; an entrance channel 1,500 ft long, 100 ft wide increasing to 200 ft at the lakeward end, and bottom elevation of -5.5 ft LWD from the lake to the center of a 200-ft-long channel transition area; and a channel 100 ft wide with a bottom elevation of -3.5 ft LWD from the channel transition area upstream approximately 3,500 ft. These project depths result in a minimum summer navigation depth of 8 and 6 ft of water, respectively, based upon water levels at the time of design. The bottom elevation was to be adjusted based

upon long-term lake level changes. A variable channel bottom elevation was adopted as maintenance would be more expensive with project depths at -8 and -6 ft LWD during high lake levels because, with the high sediment discharge rate of Cattaraugus Creek, the deeper channel would create a more efficient sediment trap.

21. The breakwaters at the site are of rubble-mound construction. The south breakwater has a massive concrete cap, which provides a walkway for fishermen at an elevation of +12.5 ft LWD. Side slopes for the entire structure are 1V:2H. The design lake level was +8.0 ft LWD, which has a 10-year recurrence interval. The height of the 20-year design nonbreaking wave (H_s) at the south breakwater head is 12.4 ft, its period (T_p) is 8.9 sec, and its direction is from the northwest. The design wave for the north breakwater is decreased to 8.9 ft because of shallower water depths and protection afforded by the south breakwater. Armor stone for the south breakwater ranges from 2.0 to 5.0 tons at the shore end to 6.0 to 13.0 tons at the structure head, and for the north breakwater from 2.0 to 5.0 tons, as indicated in Table 3.

22. It should be noted that the range of armor (0.9 to 2.0 W, where W is the weight of the individual armor unit) used in North Central Division is different from the range recommended in the Shore Protection Manual (1984) of 0.75 to 1.25 W. No additional gradation requirements are imposed upon the range of 0.9 to 2.0 W. The modified range has resulted from experience and has been determined to result in a more economical structure based upon quarry operating procedures.

23. The south breakwater is rooted on a sand dune at elevation +12.5 ft LWD, extends lakeward on a 330-deg azimuth to Sta 12+50, and thence curves toward the north, tending to parallel the shore near the head section at Sta 18+50, where the lake bottom is at -9 ft LWD. The purpose of the alignment is to provide adequate protection from the large storm waves approaching from the southwest quadrant and to prevent the predominant northward littoral drift from entering the harbor. The northward facing entrance allows the sediment load of the creek to deposit in shallow water where it may be transported shoreward by wave action to the downdrift littoral zone.

24. The north breakwater is rooted on a sand spit whose elevation is about +8 ft LWD, and it extends 600 ft lakeward on a 307-deg azimuth to its head where the lake bottom is at -7 ft LWD. The breakwater crest elevation is +12.5 ft. Lying in the shadow of the south breakwater for most of the

Table 3

Design Wave and Armor Weights

Structure	Section	Incident Wave H_s ft	Incident Wave T_p sec	Wave Type	Unit Armor Weight, lb	Calculated Armor Weight Range: 0.9W-2.0W tons	Adopted Armor Weight Range tons
South breakwater	18+50 to 11+00	12.4	8.9	Nonbreaking	12,736	5.7-12.0	6.0-13.0 (17+00 to 18+50)
	18+50 to 11+00	12.4	8.9	Nonbreaking	8,915	4.0- 9.0	4.0-9.0 (11+00 to 17+00)
	11+00 to 9+00	12.0	8.9	Breaking	9,234	4.2- 9.2	4.0-9.0
	9+00 to 2+50	7.6	8.9	Breaking	2,346	1.1- 2.4	2.0-5.0
	2+50 to 0+00	6.7	8.9	Breaking	1,607	1.0- 1.6	2.0-5.0
North breakwater	6+00 (Head)	8.9	7.2	Nonbreaking	4,709	2.1- 4.7	2.0-5.0
	6+00 to 3+00	9.3	7.2	Nonbreaking	3,761	1.7- 3.8	2.0-5.0
	3+00 to 2+00	9.3	7.2	Nonbreaking	3,761	1.7- 3.8	2.0-5.0
	2+00 to 0+00	8.4	7.2	Nonbreaking	3,761	1.7- 3.8	2.0-5.0

incident waves, this breakwater is exposed to less severe wave impact. Its primary purpose is to prevent littoral drift and windrowed ice from blocking the navigation channel. Although the net littoral transport is from south to north, reversals of drift direction would tend to form a fillet of sand at the creek mouth. Hydraulic model tests showed that ice jamming between the breakwater heads should not be a problem.

25. Profiles taken through the north and south breakwaters indicated that the ground line abutted underlayer and armor layer stones. These layers have large voids which, with the aid of wave action, would permit transport of littoral drift through the breakwaters. The underlayer and armor layers were rendered sand tight in these areas by placing a filter cloth membrane along the axis of each structure (Figures 16 and 17). The core material is sufficiently impermeable to sand and prevents its passage through the breakwater in the deeper water areas. The filter material was placed along breakwater stations that would be abutted by a fillet as it accretes.

26. The north breakwater was rooted to a low sand spit subject to erosion by lake waves and creek currents. An armored berm along 550 ft of this spit was designed to connect the root of the north breakwater to higher ground. The berm has the same size armor as the north breakwater and the same crest elevation. The berm was excavated to 0.0 ft LWD to protect against potential shoreline retreat during episodes of deficient littoral nourishment along the downdrift beach.

27. The benefit cost ratio for this project was 2.2 to 1 with a project life of 50 years. The design was completed on 31 January 1978. The New York State Office of Parks and Recreation executed the Local Cooperation Agreement on 3 July 1981. Plans and specifications were approved on 16 May 1980 and a construction contract awarded 18 September 1981. Construction was completed January 1983.

Monitoring Completed Coastal Projects Program

28. The navigation project at Cattaraugus Creek was selected for monitoring under the Monitoring Completed Coastal Projects (MCCP) Program during the program's third year, 1983. The program has as its goal the advancement of coastal engineering technology. It is designed to determine how well projects are accomplishing their purposes and are resisting the attacks of the physical environment. Those determinations, combined with

concepts and understanding already available, will lead to upgrading the credibility of predictions of cost effectiveness of engineering solutions to coastal problems; to strengthening and improving design criteria and methodology; to improving construction practices; and to improving operation and maintenance techniques. Additionally, the monitoring program will identify concerns that laboratories should address more intently. Stated in another way, the objective is the advancement of the engineering science derived from insights into the physics that laboratory studies have developed.

29. To develop the direction for the MCCP Program, the Corps of Engineers (CE) established an Ad Hoc Committee of coastal engineers and scientists. The committee formulated the program's objectives, developed its operational philosophy, recommended funding levels, and established criteria and procedures for project selection. A significant result of their efforts was a prioritized listing of problem areas to be addressed, essentially a listing of the program's areas of interest (Table 4). The initial list compiled had only the first 20 items. As the program has grown, so has the list.

Table 4

MCCP Program Areas of Interest

Shoreline and nearshore current response to coastal structures.
Wave transmission by overtopping.
Prediction of controlling cross section at inlet navigation channels.
Wave attenuation by breakwaters (submerged and floating).
Bypassing at jettied and unjettied inlets.
Wave refraction and steepening by currents.
Beach-fill project monitoring.
Stability of rubble structures--investigations to determine causes of failure.
Comparison of pre- and postconstruction sediment budgets.
Wave and current effects on navigation.
Dynamics of floating structures.
Wave reflection.
Effects of construction techniques on scour and deposition near coastal structures.
Diffraction around prototype structures.
Wave runup on structures.
Onshore/offshore sediment movement near coastal structures.
Harbor oscillations.
Wave transmission through structures.
Material life cycle.
Ice effects on structures and beaches.
Model study verification.
Wave translation.
Construction techniques.

30. The selection process has worked well since the first projects were nominated in 1981. Periodically, the CE coastal offices are invited to nominate projects for monitoring under the program. Nominations are reviewed and prioritized by a selection committee composed of representatives from Headquarters, US Army Corps of Engineers (HQUSACE), the Coastal Engineering Research Center (CERC), WES; and the CE coastal Division offices. Final selection is based on the prioritized list of projects and funding available.

31. While guidance is provided by HQUSACE, management of the program rests with CERC. Operation of the program, though, is a cooperative effort between CERC and the individual CE District offices. Development of the monitoring plan and the conduct of data collection depend on the combined resources of CERC and the Districts.

PART II: MONITORING PROGRAM

Data Collection

32. Data collection lasted from mid-1983 until the end of 1985; also, several inspections were performed after the formal monitoring period that will be discussed in this report. During the slightly more than 2 years of monitoring, significant data were collected.

33. Because of the nature of the monitoring effort, photography played an important part in evaluating project success. Controlled color aerial photography at a scale of 1 in. = 400 ft was obtained of the shoreline adjacent to the project and upstream as far as the Thruway Bridge. Dates of the photography were 18 December 1982, prior to the official monitoring but used in the evaluation of the project; 15 May 1983; 2 September 1983; 16 May 1984; and 25 June 1985.

34. Black and white enlargements, 1 in. = 10 ft, were made of the aerial photographs covering the breakwaters. These were used to identify any significant changes in the armor layer and were made from the spring photography each of the 3 years.

35. A videotape was made to document the mechanism of the ice flow breakup on 13 February 1984. Still photographs of the breakup on 25 February 1985 were obtained from a helicopter.

36. To identify any motion of the armor during the period of monitoring, structural surveys and underwater inspections were conducted. Twice a year, surveys were made of points on the breakwater walkways and selected armor stones, typically one near the crest and one near the waterline on each survey line. These survey lines actually extended over the structure profile to document any scour of the berm or toe Structural surveys were obtained in May-June 1983, September-October 1983, July-August 1984, and June 1985.

37. As mentioned, underwater visual inspections of the breakwaters were included in the monitoring effort. The inspection of the north breakwater was completed in July 1983 and the south in September 1984.

38. Profile surveys of the project area and adjacent shoreline were acquired twice a year in the spring and fall. The purpose of the surveys was to document the shoreline response to the harbor improvements and to verify the sediment transport pattern predicted by the design level littoral evaluation and fixed-bed-with-tracer modeling technique. The navigation channel was

also surveyed, although only once a year. Sediment samples were collected along many of the profile lines shown on Figure 18 and were analyzed.

39. Beach and offshore surveys were conducted in October-November 1982, May-June 1983, September-October 1983, July-August 1984, and June-July 1985. Soundings of the navigation channel were obtained in December 1982-January 1983 after contract dredging, May 1983, October 1983, July 1984, and July 1985. Sediment samples were obtained in June and October 1983 and August 1985.

40. Two nondirectional, SeaData 635-11 wave gages were installed on one tripod, for redundancy, approximately 1,000 ft north of the south breakwater head in an average water depth of 21 ft. Wave data were collected during the ice free period from 26 April to 12 December 1983. Littoral Environment Observations (LEO) were made at two sites, one on each side of the harbor. Reasonable data were acquired from the observer on the south side in 1983 and from the north side in 1984. Data on the north side were somewhat limited in 1983 and quite sparse in 1985, limited only to the summer months. On the south side, there were very limited data for 1984 and 1985, again limited to the summer months.

41. Site inspections were conducted during floods to document the effectiveness of the structures in preventing flood damage. Ground level photography was acquired during these visits. Periodic inspections were made of the structures at other times throughout the year as well.

Results

Waves

42. Figures 19 and 20 show the history of significant wave height and peak frequency, the frequency associated with the peak of the energy spectrum, for the period from 1 October 1983 until the gage was removed in December. While there were sev. -l storms during this period, the most significant was 13-14 October 1983. Spectra for that storm are shown in Figures 21 through 26. Prior to the 2100 data collection on 13 October, there was little energy in the spectra. The highest significant wave height calculated during the storm was 7.7 ft with a peak period of 8.26 sec. This was the highest significant wave height experienced during the wave data collection, considerably less than the incident wave height of 12.4 ft used in the design.

43. LEO data were a bit too sparse to provide any real information for the project, but some of the information is worth reporting. Both the observers reported maximum wave heights of 5 ft or better, with the observer on the north beach reporting 5.5 ft. The longest period reported was by the south beach observer, at 11.0 sec. A maximum period of only 5.6 sec was reported north of the harbor. Both observers reported over 70 percent of the waves between 0 and 1.9 ft high and at periods between 0 and 4.9 sec; but above a wave height of 1.9 ft, the observer on the north side saw more large waves than the person to the south. The north side observer saw 98 percent of the waves with heights of 4.9 ft or less. On the south side, 98 percent of the waves were reported as not more than 3.9 ft.

44. On 2 December 1985, there was a significant storm that was thought to be near the design storm. Unfortunately neither gage nor LEO data were available for that storm. To estimate the storm wave parameters, a desk study was conducted at USAED, Buffalo, to hindcast the storm waves. That study produced an estimate of 12.5 ft and 8 sec for the deepwater wave, resulting in an H_s at the head of the south breakwater of 8.7 ft, again well below the design wave height.

Structure stability

45. Surveys of selected stones along the breakwaters, in addition to structure cross sections, were conducted in May-June 1983, September-October 1983, July-August 1984, and June 1985 to determine any structural settlement. Along the south breakwater, survey points (targets) were established at Sta W-6 (head), W-5, W-4, W-3, W-2, Sta 57+00, Sta 59+00, and Sta 64+00. Along the north breakwater, survey points were established at Sta 54+87.80 (head), 56+00, 57+00, and 59+00. These points are located along the center line of the breakwater walkway. At these locations, on each side of the breakwater, a point was established on a rock at the waterline and one at the crest. At the structure head, five sections were taken at 90 deg, 45 deg, and along the axis line of the breakwater crest. Each point was surveyed and referenced to its vertical and horizontal position. A change in position was expressed in cylindrical coordinates (Δr, $\Delta\theta$, Δz), with a positive Δr, indicating stone movement outward from the center line of the breakwater, a positive $\Delta\theta$ indicating a counterclockwise rotation about the survey point on the breakwater center line, and a positive Δz indicating stone upheaval. Figure 27 presents the station locations along the breakwater and a sketch of the surveyed stone locations. Table 5 presents the location, elevation, and

Table 5

Target Locations and Changes in Elevation Along Breakwaters

Target Locations*							Change in Elevation, ft**			
Line	Bearing	Distance ft	Point	North Coordinate	East Coordinate	Elevation	May-Jun 1983 to Sep-Oct 1983	Sep-Oct 1983 to Jul-Aug 1984	Jul-Aug 1984 to Jun 1985	May-Jun 1983 to Jun 1985
						South Breakwater				
			W1	936,196.28	350,979.03					
W1 to 64+00	N37-44-44.5W	414.74	64+00	936,524.22	350,725.15	+12.14	0.30	0.03	0.01	0.34
64+00 to 59+00	N37-44-44.5W	501.87	59+00	936,921.07	350,417.92	+12.53	0.02	0.02	0.02	0.06
59+00 to 57+00	N37-44-44.5W	200.75	57+00	937,079.81	350,295.03	+12.47	0.01	0.0	0.03	0.04
57+00 to W2	N37-44-44.5W	225.22	W2	937,257.90	350,157.16	+12.50	0.01	-0.01	0.12	0.12
W2 to W3	N30-43-38.9W	105.69	W3	937,348.75	350,103.16	+12.49	0.01	-0.02	0.15	0.14
W3 to W4	N20-19-39.6W	120.28	W4	937,461.54	350,061.38	+12.49	0.01	0.03	0.01	0.05
W4 to W5	N07-51-56.2W	165.65	W5	937,625.63	350,038.71	+12.34	0.02	-0.02	0.11	0.11
W5 to W6	N07-48-22.4E	177.72	W6	937,801.70	350,062.85	+12.30	0.0	0.10	-0.04	0.06
						North Breakwater				
			X	937,240.10	351,033.84					
X to 59+00	N73-21-32W	212.43	59+00	937,300.93	350,830.31	+12.58	-0.06	0.0	0.0	-0.06
59+00 to 57+00	N50-43-32W	210.12	57+00	937,433.95	350,667.65	+12.49	0.01	-0.02	0.03	0.02
57+00 to 56+00	N50-43-32W	105.32	56+00	937,500.62	350,586.12	+12.58	-0.01	0.03	-0.03	-0.01
56+00 to	N50-43-32W	118.08	54+87.80	937,575.37	350,494.71	+12.55	0.0	0.02	-0.04	-0.02

* Target locations did not change from May-June 1983 to June 1985.
** Negative value indicates target settlement.

change in elevation of the target locations on the south and north breakwaters. Tables 6 and 7 present the measured stone movement. Figures 28 through 36 present structure cross sections for the south breakwater along the trunk (Sta 57+00) and the head (W-6) and the north breakwater head (Sta 54+87.8).

46. The horizontal location of the targets on the center line of the breakwater walkways experienced no change over the period May-June 1983 to June 1985, indicating the breakwaters had not shifted in position. However, the south breakwater experienced a small upheaval ranging from 0.04 to 0.34 ft, with an average change of 0.12 ft. The north breakwater targets experienced a change in elevation of -0.06 to 0.02 ft with an average settlement of 0.02 ft.

47. Stones along the lakeside of the south breakwater (points A and B on Figure 27) moved perpendicular to the breakwater center line -0.11 to 0.15 ft with an average movement inward of 0.01 ft. The stones rotated about the target -1.33 to 0.2 deg for an average clockwise movement (lakeward along breakwater axis) of 0.23 deg. Vertical stone movement was -0.1 to 0.2 ft with an average upheaval of 0.05 ft. The harborside stones (C and D, Figure 27) on the south breakwater moved perpendicular to the breakwater center line -0.12 to 0.24 ft for an average outward movement of 0.03 ft, rotated about the target -0.18 to 0.17 deg with an average clockwise movement of 0.03 deg, and moved vertically 0 to 0.2 ft with an average upheaval of 0.13 ft. At the breakwater head (W-6), stone radial movement from the target was -0.11 to 0.86 ft, rotation experienced about the target was -1.72 to 1.63 deg, and vertical movement was -0.6 to 0 ft. Measurements were not obtained for stone W-6B, which was covered; stone W-6D, which was not found in June 1985; and stone W-6F, which was on its side by summer 1984.

48. These values indicate negligible movement along the structure trunk. Stone W-2C moved outward 0.92 ft between September-October 1983 and July-August 1984 but moved inward 0.92 ft between July-August 1984 and June 1985, with negligible rotation or change in elevation. Stone W-2D also moved outward 3.46 ft from May-June 1983 to September-October 1983 and moved inward 3.44 ft the subsequent survey period, with negligible rotation or change in elevation. This suggests a surveying error. Structure cross sections taken along the trunk also indicate negligible movement. Figures 28 and 29 present cross sections of the structure trunk at Sta 57+00. Stone survey data of the head indicate minor damage has occurred. Cross sections of the head section

Table 6

Change in Stone Location, South Breakwater

Paint On Stone Locations, South Breakwater							May-Jun 1983 to Sep-Oct 1983			Sep-Oct 1983 to Jul-Aug 1984			Jul-Aug 1984 to Jun 1985			May-Jun 1983 to Jun 1985		
Line	Bearing	Distance ft	Point	North Coordinate	East Coordinate	Elevation	Δr ft	Δθ deg	Δz ft	Δr ft	Δθ deg	Δz ft	Δr ft	Δθ deg	Δz ft	Δr ft	Δθ deg	Δz ft
			64+00	936,524.22	350,725.15	+12.14												
64+00 to A	S52-15-15.5W	5.66	64+00A	936,520.76	350,720.67	+13.2	-0.01	0	0	0	0	0	0	0.18	0.1	-0.01	0.18	0.1
64+00 to B	S52-15-15.5W	32.89	64+00B	936,504.09	350,669.14	+5.8	-0.08	-0.05	0.1	0	-0.07	-0.1	0.01	0.08	0.1	-0.07	-0.03	0.1
64+00 to C	N52-15-15.5E	10.82	64+00C	936,530.84	350,733.71	+11.6	0.02	-0.05	0.1	0.1	-0.02	-0.01	-0.01	-0.03	0.1	0.02	-0.12	0.1
64+00 to D	N52-15-15.5E	24.73	64+00D	936,539.36	350,744.70	+4.4	0.08	-0.08	0.1	0.02	0	0	0	0.03	0	0.1	-0.05	0.1
			59+00	936,921.07	350,417.92	+12.53												
59+00 to A	S52-15-15.5W	3.69	59+00A	936,918.81	350,415.00	+13.5	0	0.05	0.1	0.01	-0.27	0	-0.01	0.03	0	0	-0.18	0.1
59+00 to B	S52-15-15.5W	29.36	59+00B	936,903.10	350,394.70	+6.0	-0.08	0	0	0.08	-0.15	0	-0.11	--	0	-0.11	--	0
59+00 to C	N52-15-15.5E	11.14	59+00C	936,927.89	350,426.73	+11.1	0.02	-0.12	0	0.01	0	0	-0.15	-0.07	0	-0.12	-0.18	0
59+00 to D	N52-15-15.5E	20.49	59+00D	936,933.61	350,434.12	+7.5	0.03	-0.02	0.1	-0.01	-0.12	0	0	0.05	0	0.02	-0.08	0.1
			57+00	937,079.81	350,295.03	+12.47												
57+00 to A	S52-15-15.5W	5.01	57+00A	937,076.74	350,291.07	+12.6	0.01	0.02	0	0.01	-0.1	0	0	-0.1	0	.02	-0.18	0
57+00 to B	S52-15-15.5W	19.87	57+00B	937,067.65	350,279.32	+5.5	0.06	0.08	0.1	-0.10	-0.07	0	-0.01	-0.1	0	-0.05	-0.08	0.1
57+00 to C	N52-15-15.5E	6.85	57+00C	937,084.00	350,300.45	+15.1	-0.02	-0.02	0	-0.01	-0.02	0	-0.01	0.2	0	-0.02	0.17	0
57+00 to D	N52-15-15.5E	26.23	57+00D	937,095.87	350,315.77	+7.0	0.07	-0.07	0.1	0.01	-0.02	0	0	-0.03	0	0.08	-0.12	0.1
			W2	937,257.90	350,157.16	+12.50												
W2 to A	S52-15.15.5W	6.09	W2A	937,254.17	350,152.34	+12.1	-0.02	-0.5	0	0.01	-0.07	0	0	-0.05	0.1	-0.01	-0.61	0.1
W2 to B	S52-15-15.5W	15.89	W2B	937,248.17	350,144.60	+7.4	0.05	-0.05	0.1	-0.04	-0.08	0	-0.04	-0.05	0.1	-0.03	-0.18	0.2
W2 to C	N52-15-15.5E	5.97	W2C	937,261.55	350,161.88	+11.9	-0.02	-0.13	0	-0.92	0.03	0.1	0.92	0.27	0.1	-0.02	0.17	0.2
W2 to D	N52-15-15.5E	22.84	W2D	937,271.88	350,175.22	+7.0	3.46	0.1	0.1	-3.44	0.1	0	0.08	-0.1	0.1	0.10	0.1	0.2
			W3	937,348.75	350,103.16	+12.49												
W3 to A	S64-28-20.4W	4.04	W3A	937,347.01	350,099.51	+12.5	0.04	0.53	0	-0.01	-0.22	0	0.01	-0.12	0.2	0.04	0.2	0.2
W3 to B	S64-28-20.4W	17.72	W3B	937,341.11	350,087.17	+7.0	0.03	0.07	0	0.1	-0.15	-0.1	0.02	0.1	0	0.15	0.02	-0.1
W3 to C	N64-28-20.4E	6.15	W3C	937,351.40	350,108.71	+13.5	-0.02	-0.12	0.1	0.01	0.02	0	0.01	-0.03	0.1	0	-0.13	0.2
W3 to D	N64-28-20.4E	19.83	W3D	937,357.30	350,121.05	+9.2	0	-0.05	0	0.02	0	0.1	-0.01	0.02	0.1	0.01	-0.03	0.2
			W4	937,461.54	350,061.38	+12.49												
W4 to A	S75-54-03.8W	5.92	W4A	937,460.10	350,055.64	+11.3	0.01	-0.05	0	0.02	0	0	-0.02	-1.28	-0.1	0.02	-1.33	-0.1
W4 to B	S75-54-03.8W	14.37	W4B	937,458.04	350,047.44	+8.4	0.01	-0.03	0.1	-0.01	-0.07	-0.1	-0.06	0.02	0.1	-0.06	-0.08	0.1
W4 to C	N75-54-03.8E	10.47	W4C	937,464.09	350,071.53	+13.0	0.01	-0.1	0	-0.03	0.02	0.1	-0.04	-0.02	0.1	-0.06	-0.1	0.2
W4 to D	N75-54-0.38E	28.86	W4D	937,468.57	350,089.37	+4.4	0.11	-0.05	0	-0.06	0.03	0.1	0.19	0.08	0	0.24	0.07	0.1
			W5	937,625.63	350,038.71	+12.34												
W5 to A	S89-59.22.4W	5.86	W5A	937,625.63	350,032.85	+11.6	0.03	-0.05	0	-0.7	0.05.	-0.1	0.70	-0.23	0.1	0.03	-0.23	0
W5 to B	S89-59-22.4W	17.89	W5B	937,625.63	350,020.82	+8.4	-0.07	-0.07	0	-0.04	-0.17	-0.1	-0.04	-0.2	0	-0.07	-0.43	-0.1
W5 to C	N89-59-22.4E	6.13	W5C	937,625.63	350,044.84	+12.2	0	0	0	0.02	-0.03	0	-0.02	0.02	0.1	0	-0.02	0.1
W5 to D	N89-59-22.4E	22.86	W5D	937,625.63	350,061.57	+7.2	-0.06	-0.05	0.1	0.02	0.03	0	0.07	-0.03	0.1	0.03	-0.05	0.2
			W6	937,801.70	350,062.85	+12.30												
W6 to A	N82-11-37.6W	4.13	W6A	937,802.26	350,058.76	+12.4	0.01	-0.37	0.1	-0.08	-1.15	0	-0.04	0.03	-0.1	-0.11	1.47	-0.2
W6 to B	N82-11-37.6W	16.69	W6B	937,803.97	350,046.31	+7.5	-0.01	0.02	0	Covered			Covered			Covered		
W6 to C	S82-11-37.6E	9.57	W6C	937,800.40	350,072.33	+12.1	0.02	-0.07	0	0.06	0.12	0.1	0.07	-0.57	-0.1	0.15	-0.52	0
W6 to D	S82-11-37.6E	26.49	W6D	937,798.10	350,089.09	+4.2	-0.16	0.05	0	0.26	0.02	0.1	Not Found in June 1985					
W6 to E	N37-11-37.6W	6.25	W6E	937,806.68	350,059.07	+13.1	0	0	0	0.04	-7.08	0	-0.04	5.33	-0.1	0	-1.72	-0.1
W6 to F	N37-11-37.6W	23.42	W6F	937,820.36	350,048.69	+6.3	-0.2	-0.02	0.5	-0.48	0.87	0.5	Stone on Side			Stone on Side		
W6 to G	N07-48-22.4E	6.81	W6G	937,808.45	350,063.78	+13.8	0	0	0	-0.01	0.27	0.1	-0.02	0.18	-0.1	-0.03	0.45	0
W6 to H	N07-48-22.4E	25.98	W6H	937,827.44	350,066.38	+7.1	-0.03	0	0	0.17	-0.12	0	0.03	0.45	-0.3	0.17	0.33	-0.3
W6 to I	N52-48-22.4E	7.86	W6I	937,806.45	350,069.11	+12.7	0	0	0	0.8	-0.27	0.1	0.03	0.6	-0.1	0.11	0.33	0
W6 to J	N52-48.22.4E	31.74	W6J	937,820.89	350,088.13	+7.6	-0.01	0	-0.1	0.71	0.9	-0.4	0.16	0.73	-0.1	0.86	1.63	-0.6

Table 7

Change in Stone Location, North Breakwater

Paint on Stone Locations, North Breakwater							May-Jun 1983 to Sep-Oct 1983			Sep-Oct 1983 to Jul-Aug 1984			Jul-Aug 1984 to Jun 1985			May-Jun 1983 to Jun 1985		
Line	Bearing	Distance ft	Point	North Coordinate	East Coordinate	Elevation	Δr ft	Δθ deg	Δz ft	Δr ft	Δθ deg	Δz ft	Δr ft	Δθ deg	Δz ft	Δr ft	Δθ deg	Δz ft
			59+00	937,300.93	350,830.31	+12.58												
59+00 to A	S39-16-18W	8.20	59+00A	937,294.58	350,825.12	+12.8	-0.01	-0.18	0	-0.03	0.02	0.1	0.03	-0.17	-0.1	-0.01	-0.33	0
59+00 to B	S39-16-18W	20.33	59+00B	937,285.04	350,817.31	+6.3	-0.07	0.05	0	0.05	-0.01	0	Covered by Tree					
59+00 to C	N39-16-18E	14.45	59+00C	937,312.12	350,839.46	+8.9	-0.01	-0.15	-0.5	0.04	0.13	0	-0.04	0	0	-0.01	-0.02	-0.5
			57+00	937,433.95	350,667.65	+12.49												
57+00 to A	S39-16-18W	5.61	57+00A	937,429.61	350,664.10	+12.8	-0.01	-0.03	0	-0.01	0.18	0.2	0.01	-0.12	-0.3	-0.01	0.03	-0.1
57+00 to B	S39-16-18W	20.78	57+00B	937,417.86	350,654.50	+5.4	0.04	-0.03	0.1	0.11	0.05	0	-0.11	-0.05	0	0.04	-0.03	0.1
57+00 to C	N39-16-18E	4.54	57+00C	937,437.46	350,670.52	+13.3	0.01	0.23	0	-0.03	-0.35	0	0.03	0.03	0	0.01	-0.08	0
57+00 to D	N39-16-18E	17.72	57+00D	937,447.67	350,678.87	+6.7	-0.02	0.02	0	0	0	0	0.08	-0.05	0.1	0.06	-0.03	0.1
			56+00	937,500.62	350,586.12	+12.58												
56+00 to A	S39-16-18W	7.42	56+00A	937,494.88	350,581.42	+11.9	-0.08	-0.12	0	0.07	0.13	0	0	-0.05	0	-0.01	-0.03	0
56+00 to B	S39-16-18W	[illegible]	56+00B	937,486.03	350,574.19	+5.5	0.02	-0.12	0	0.24	0.05	-0.1	-0.2	0.07	0	0.06	-0.03	0.1
56+00 to C	N39-16-18E	3.98	56+00C	937,503.70	350,588.64	+12.8	0.01	0	-0.1	-0.01	-0.03	0	-0.01	0.15	0.1	-0.01	0.12	0
56+00 to D	N39-16-18E	19.59	56+00D	937,515.79	350,598.52	+6.1	-0.01	-0.03	0	0.14	-0.1	-0.1	-0.14	0.12	0.1	-0.01	-0.02	0
			54+87.8	937,575.37	350,494.71	+12.55												
54+87.8 to A	S39-16-18W	5.23	54+87.8A	937,571.32	350,491.40	+12.4	-0.03	0.03	-0.1	-0.01	0.02	0	[illegible]	0.02	0.1	-0.03	0.07	0
54+87.8 to B	S39-16-18W	18.53	54+87.8B	937,561.02	350,482.98	+7.7	0.06	0	0	0.02	0.03	-0.1	-0.02	-0.07	0.1	0.06	-0.03	0
54+87.8 to C	N39-16-18E	6.62	54+87.8C	937,580.49	350,498.90	+11.7	-0.12	0.02	0	0.06	0.02	0	0	-0.12	0	-0.06	-0.08	0
54+87.8 to D	N39-16-18E	20.68	54+87.8D	937,591.38	350,507.80	+4.8	-0.01	0	0.1	0.10	-0.50	0	-0.1	0.48	0	-0.01	0.02	-0.1
54+87.8 to E	S84-16-18W	14.10	54+87.8E	937,573.96	350,480.68	+12.6	0.01	0.03	0	0.02	0.02	0	0.03	-0.08	0	0.06	-0.03	0
54+87.8 to F	S84-16-18W	27.20	54+87.8F	937,572.66	350,457.65	+4.9	-4.07	0	0	0.39	0.03	0	-0.11	0.03	0	-3.80	0.07	0
54+87.8 to G	N45-43-42W	14.86	54+87.8G	937,585.74	350,484.07	+11.6	0.01	0.03	-0.01	0.04	0.05	0	-0.04	0.02	0	0.01	0.01	-0.1
54+87.8 to H	N45-43-42W	27.20	54+87.8H	937,594.36	[illegible]	+4.8	-0.11	0.03	0	0.02	0.02	-0.1	-0.02	0.02	0.1	-0.11	-0.03	0
54+87.8 to I	N05-43-42W	8.59	54+87.8I	937,583.92	350,493.85	+12.8	-0.03	0.05	0	0.02	0.05	0	-0.01	-0.08	0	-0.02	0.02	0
54+87.8 to J	N05-43-42W	29.52	54+87.8J	937,604.74	350,491.76	+4.1	-0.1	0.07	0	0.08	0.12	0	--	--	--	--	--	--

(W-6) are presented on Figures 30 through 34 and indicate minor stone steepening and scour near the toe.

49. Stones along the lakeside of the north breakwater (C and D on Figure 27) moved perpendicular to the breakwater center line -0.01 to 0.06 ft for the period May-June 1983 to June 1985. The accompanying rotation about the target on the breakwater center line and change in elevation was -0.08 to 0.12 deg and -0.5 to 0.1 ft, respectively. The harborside stones along the trunk experienced values for (Δr, $\Delta\theta$, Δz) of -0.01 to 0.06 ft, -0.08 to 0.12 deg, and -0.1 to 0.1 ft, and the head experienced values of -0.11 to 0.06 ft, -0.08 to 0.07 deg, and -0.1 to 0.0 ft, respectively. Not included in these values for the head section is an immediate movement of stone 54+87.80 F (shown in Figure 27) inward 4.07 ft with no change in rotation or elevation and its subsequent return the following survey. It is surmised that an initial surveying positioning error explains this anomaly. These values in addition to the structure cross sections indicate negligible damage to the north breakwater, which has been confirmed during subsequent field inspections. Figures 35 and 36 present cross sections taken at the north breakwater head. The surveys indicate that some scouring near the toe has occurred but has not affected structural integrity.

50. In addition to the stone movement surveys and structure cross sections, dive inspections of the breakwaters were completed in 1983 and 1984. The north breakwater was inspected only in 1983 due to wave activity during the 1984 inspection. It was concluded that the structure toe is uniform and that there is no evidence of scour. The channel side of the south breakwater was inspected in 1984 with diving beginning at Sta 56+00 and progressing lakeward. From Sta 56+00 to W-2, the berm was found wide and intact and sloped down to a flat mud bottom. At W-4, the toe was steeper. Stones at the toe appeared loose and may have fallen down slightly, but there did not appear to be any loss in berm width. Between W-4 and W-5, the stone berm dropped off rapidly with some bedding exposed, but the berm was mainly intact. Between W-5 and W-6, some bedding was exposed in the toe berm, but the berm was intact. The inspections concluded that the toe is stable.

51. To alleviate the potential detrimental effects of scour at the toe of the breakwaters, a sacrificial toe berm was constructed by placing one layer of armor over the bedding material as shown on the structure cross sections presented in Figures 16 and 17. Figures 37 and 38, taken in November 1988 of the south breakwater, show the toe berm protruding slightly above the

water. Although the bedding thickness was specified at 3 ft, during construction the contractor maintained a fairly constant bedding elevation. This resulted in the bedding thickness increasing at some breakwater sections to 7 ft. The crest of the toe berm is 3 to 4 stones wide. It is theorized that if scour occurred, the toe stones would successively slide down into the scour area with the armor slope remaining intact. Despite the variation in bedding thickness, it appears that the toe berm has stabilized the bedding.

52. Negligible stone movement is only a partial indication of structure stability, as the wave climate during the period of observation must also be considered. The maximum wave height and period observed or measured during the stone monitoring period of September 1983 to June 1985 was 7.7 ft, 8.26 sec and occurred on 14 October 1983. The structure design considered the joint occurrence of a 10-year water level, 576.7 ft IGLD, which is +8.1 ft LWD, and a 20-year wave, $H = 11.5$ ft , $T = 8.9$ sec , Angle Class III (generally from west-southwest to south) for the south breakwater and $H = 6.2$ ft , $T = 7.2$ sec , Angle Class II (generally from west-northwest to west-southwest) for the north breakwater. Based upon a design significant wave height of 12.4 ft at the south breakwater head compared with the largest observed wave during the monitoring period, the structure was not tested under design conditions. The wave hindcast discussed previously for the 2 December 1985 storm estimated a significant wave height incident to the south breakwater head of 8.7 ft, which is considerably less than the design wave height of 12.4 ft. The exact cause of the minor damage at the south breakwater head is unknown as wave heights have been less than the design wave height. However, it is postulated that this may have been caused by stone cracking. As discussed in a subsequent section (Recent Observations), cracking of the armor stones at the head has been observed. Loss of a few stones by stone shattering would have caused adjacent stones to collapse into the void, resulting in a steepening of the structure slope.

Sediment transport

53. The general offshore bathymetry prior to construction of the harbor structures at the mouth of Cattaraugus Creek is presented on Figure 39. A vertical shale bluff without a beach at the base was present in Hanford Bay. Progressing north, a beach existed along the entire shore to Cattaraugus Creek. Midway between Hanford Bay and Cattaraugus Creek, the offshore contours did not follow the general shoreline configuration but made a rather dramatic bulge lakeward, a situation probably not related to modern processes

but remnant from old river channels. As will be discussed, this area experienced the least change within the monitoring area after construction of the breakwaters. A similar bathymetric pattern has been observed 13 miles south near Van Buren Point. Offshore contours generally paralleled the shoreline in this area.

54. To evaluate beach response to the presence of the harbor structures, profile surveys of the project area and the adjacent shoreline were taken November 1982, October 1983, August 1984, and July 1985. Survey lines were located at 100-ft intervals from 600 ft south of the south breakwater to 600 ft north of the north breakwater and at 500-ft intervals 3,500 ft farther south and 4,500 ft farther north, resulting in coverage of 10,000 ft of shoreline. The surveys were conducted by a combination of rod and level, lead line, and Fathometer from the back beach to 0.5 mile offshore. Figures 40-48 present comparative profiles for Sta 450+00, 465+00, 485+00, 490+00, 495+00, 501+00, 505+00, 525+00, and 550+00, respectively. Figures 49 and 50 present aerial views of the area near the breakwaters on 13 May 1983 and 26 October 1985.

55. The profiling information revealed that at the south end of the beach monitoring area, Sta 450+00, minor erosion of the profile occurred above -6 ft LWD with accretion occurring below. In the central area of Sunset Bay, where the offshore contour bulge exists, little change has occurred. This is the only area where a pronounced 5-ft bar exists offshore. In the fillet area of the south breakwater, accretion has occurred above 0.0 ft LWD with 1 to 2 ft of erosion occurring farther offshore. A scour hole has appeared off the south breakwater head and is approximately 800 ft in diameter with a maximum depth 4 ft lower than the surrounding areas. This change in offshore bathymetry in the vicinity of the breakwaters is readily apparent in Figure 51. It is surmised that it is caused by increased wave activity due to reflection off the south breakwater head. It should be noted, however, that the scour hole generally filled in during the period from June 1985 to June 1989. This change will be discussed further in a subsequent section (Recent Observations).

56. During the first year after completion of the project, a large area of accretion adjacent to the north breakwater occurred indicating fillet growth. A large backshore berm developed increasing the beach elevation up to 4 ft in some areas. This pattern continued at a reduced rate the following year. During the third year, minor erosion of the fillet occurred as its

orientation changed and became more oblique with the shore. North of the fillet, Sta 510+00 to 525+00, the entire profile generally accreted 1 to 3 ft with the most accretion occurring offshore. At the north end of the monitoring area, accretion has mainly occurred offshore with minor erosion of the above water profile.

57. Beach inspections were conducted on 8 June 1983, 27 June 1984, and 26 July 1985, during which measurements of the beach width using a tape from known locations to the waterline and beach characteristics were obtained. Table 8 presents the measured beach widths. The beach widths were not

Table 8

Measured Beach Widths

Station	8 June 1983 572.39 ft IGLD*	27 June 1984 572.50 ft IGLD	26 July 1985 572.99 ft IGLD
1N	(1)**	(1)	(1)
2N	(1)	(1)	(1)
3N	-	201	179
4N	130	115	92
5N	91	115	95
6N	48	51	57
7N	27	37	28
8N	94	109	73
9N	84	99	81
1S	(1)	(1)	(1)
2S	(1)	(1)	(1)
3S	127	68	62
4S	63	68	62
5S	88	95	87
6S	104	115	119
7S	34	55	37
8S	38	44	35
9S	84	94	87
10S	44	52	41

* IGLD = 568.6 ft = LWD.
** (1) = Station located on breakwater.

adjusted for water level variation, which was a maximum difference of 0.5 ft vertically. From June 1983 to June 1984, beach widths increased at all stations except for Sta 3S, which is in the fillet area of the south breakwater. This discrepancy is not explained as the shore was observed to prograde in that time period at Sta 490+00. Between June 1984 and July 1985, beach widths generally decreased on the order of 10 to 20 ft.

58. Bathymetric volume changes were computed for the five sets of beach profiles by using the Beach Profile Analysis System computer program (Fleming and DeWall 1982). Tables 9-11 present the volumetric change per survey line and by area (i.e. south of the project, the project area, and north of the project); and Figure 52 presents the average annual volume change between November 1982 and July 1985. In general, the monitoring area south of the project experienced an average gain of 46,308 cubic yards/year; offshore of the breakwaters, there was an average loss of 12,655 cubic yards/year; and north of the project, there was an average gain of 64,078 cubic yards/year, for an average annual gain of 97,731 cubic yards/year within the monitoring area.

59. The south end of the monitoring area experienced the largest gain south of the project. Progressing north, the deposition decreased as the fillet area of the south breakwater is approached. The fillet area gained significantly less than the south end of the monitoring area. The loss offshore of the project is explained by the scour hole that developed off the south breakwater head. Immediately north of the project a loss was recorded, although it is somewhat misleading. The fillet adjacent to the north breakwater appears stable, but its orientation to the incoming waves is adjusting. Significant losses farther offshore occurred as part of the generalized scour off the south breakwater head. Significant increases occurred farther north, although most of the gain was offshore. The minimum gains were experienced from Sta 530+00 to Sta 540+00.

60. Surface sediment samples were collected within a 2,500-ft grid centered at the harbor entrance along survey profile lines north, south, and offshore of the project area during May-June 1983, September-October 1983, and June 1985. Samples at 6 sites along the channel, discussed in a subsequent section, were also obtained for a total of 42 sample sites. Table 12 presents a classification summary of the sediment samples with Figures 53-66 presenting the gradation curves at selected sample sites.

Table 9

Offshore Volume Changes South of Project

		Volume Change in Cubic Yards				
Station	Width ft	Nov 1982-Jun 1983	Jun 1983-Oct 1983	Oct 1983-Aug 1984	Aug 1984-Jul 1985	Total Nov 1982-Jul 1985
450+00	250	4,110	2,710	10,625	468	17,913
455+00	500	13,015	965	22,170	-2,097	34,053
460+00	500	23,669	727	28,753	-4,026	49,123
465+00	500	559	989	7,577	927	10,052
470+00	500	2,442	10,118	18,209	-23,554	7,215
475+00	500	-4,285	790	13,332	-7,741	2,096
480+00	400	4,504	-6,628	10,792	-12,394	-3,726
483+00	250	-	-1,939	10,528	-11,340	-2,751
485+00	100	1,351	-1,149	2,835	-73	2,964
486+00	100	2,385	-697	3,629	-170	5,147
487+00	100	1,742	-692	3,640	-3,677	1,013
488+00	100	2,288	-1,048	3,282	-3,066	1,456
490+00	100	-480	990	695	-1,153	52
491+00	100	-386	-348	247	-616	-1,103

Total 123,504 yd^3
(46,308 yd^3/year)

Table 10

Volume Changes South of Project

		Volume Change in Cubic Yards				
Station	Width ft	Nov 1982-Jun 1983	Jun 1983-Oct 1983	Oct 1983-Aug 1984	Aug 1984-Jul 1985	Total Nov 1982-Jul 1985
492+00	100	-4,223	-1,584	1,388	-5,163	-9,582
493+00	100	-2,839	-995	500	-3,691	-7,025
494+00	100	-4,762	-2,710	-436	-2,220	-10,128
495+00	100	-2,225	-2,719	-2,287	-1,029	-8,360
496+00	100	-924	-1,436	367	1,200	-793
497+00	100	-1,090	-2,036	3,082	806	762
498+00	100	-1,334	-2,487	2,988	1,668	835
499+00	150	-1,128	-3,713	3,596	1,785	540

Total -33,751
(-12,665 yd^3/year)

Table 11

Offshore Volume Changes North of Project

		Volume Change in Cubic Yards				
Station	Width ft	Nov 1982- Jun 1983	Jun 1983- Oct 1983	Oct 1983- Aug 1984	Aug 1984- Jul 1985	Total Nov 1982- Jul 1985
501+00	100	5,194	-962	6,321	-13,183	-2,630
502+00	100	898	30	6,208	-9,671	-2,535
503+00	100	-105	-1,257	8,208	-11,833	-4,987
504+00	100	-2,553	-444	9.024	-7,993	-1,966
505+00	300	-8,378	-2,650	24,111	-5,574	7,509
510+00	500	-8,142	-1,461	28,581	3,349	22,327
515+00	500	-1,574	-6,890	59,078	-8,598	42,016
520+00	500	-7,012	-5,994	58,883	-7,602	38,275
525+00	500	10,392	-6,364	52,974	-23,512	33,490
530+00	500	-96	-4,985	23,483	-15,908	2,494
535+00	500	2,293	-3,009	16,208	-13,591	1,901
540+00	500	138	-4,174	21,908	-13,688	4,184
545+00	500	476	6,864	8,179	10,451	25,970
550+00	250	2,121	-1,530	4,256	-	4,847
						Total 170,895
						(64,078 yd^3/year)

61. In general, sand exists from the shore to 1,500 ft offshore with a decrease in sediment size with distance from shore. Little change in gradation during the monitoring period occurred for all offshore sample sites except for Sta 501700, 501900, 5051200, and 5101300. These sites experienced a general increase in coarseness, changing from sand to a sandy gravel except for Sta 501900, which became finer with time. It is surmised that this increase is due to deposition of sediments from Cattaraugus Creek based upon the location of the samples in the path of the creek's plume, as well as the change in gradation at channel sediment Sta 55220. Prior to construction of the project, coarse sediment from the creek was deposited near the mouth. Since project construction, it is now deposited farther offshore. It has also been noted by some of the residents that there has been an increase in debris along the north beach since the project was constructed. With the creek discharging farther offshore, some debris that would have been deposited at the mouth on the bar may now be carried out into the lake and subsequently transport landward by onshore winds.

Table 12

Sediment Sample Classification

Station	Date Sediment Sample Was Taken		
	May-Jun 1983	Sep-Oct 1983	Jun 1985
55220	GM Silty Sandy Gravel/ Traces of Organic	OH Organic Sandy Clay	ML Organic Sandy Silt
60250	SM Sandy Silt/Traces of Organic	OH Organic Sandy Clay	ML Organic Sandy Silt
65340	ML Sandy Silt/Traces of Organic	OH Organic Gravelly Sandy Clay	SM Organic Silty Sand
70230	ML Sandy Silt/Traces of Organic	OH Organic Sandy Clay	ML Sandy Silt
75160	ML Sandy Silt/Traces of Organic	OH Organic Clay	ML Organic Sandy Silt
87200	OL Organic Clay	OH Organic Clay	ML Organic Sandy Silt
480500	SP Sand	SP Sand	SP Sand
4801000	SP-SM Sand	SP Sand	SP-SM Sand
483300	SP Sand	SP Sand	SP Sand
483600	SP Sand	SP Sand	SP Sand
4831200	SP-SM Sand	SP Sand	SP-SM Sand
487100	SP Gravelly Sand	SP Gravelly Sand	GP Sandy Gravel
487300	SP Sand	SP Sand	SP Sand
487600	SP Sand	SP Sand	SP Sand
487900	SP Sand	SP Sand	SP Sand
4871300	SP Sand	SP-SM Sand	SP-SM Sand
491300	SP Sand	SP-SM (Traces of Vegetation)	SP Sand
491500	SP Sand	SP Sand	SP Sand
491800	SP Sand	SP Sand	SP Sand
4911100	SP Sand	SP Sand	SP Sand
4911500	SP Sand	SP-SM Sand	SP Sand
495200	SP-SM Sand	SP Sand	SP Sand
495400	SP Sand	SP Sand	SP Sand
495700	SP Sand	SP Sand	SP Sand
4951000	SP-SM Sand	SP-SM Sand	SP-SM Sand
498300	SP Sandy Gravel	SP-SM Sand	SP Sand
498600	SP Sand	SP-SM Sand	SP Sand
501200	GW Sandy Gravel	SP Gravelly Sand	SP Gravelly Sand
501400	SP Sand	SP Sand	SP Sand
501700	SP Sand	SP Sand	SP Gravelly Sand
501900	SP Sand	SP Sand (Traces of Vegetation)	SM Organic Silty Sand
5011200	SP Sand	SP Sand	SM Silty Sand
5011500	SP-SM Sand	SM Silty Sand	SM Silty Sand
505200	GP Sandy Gravel	GP Sandy Gravel	GW Sandy Gravel
505400	SP-SM Sand	SP Sand	SP-SM Sand
505700	SP Sand	SP Sand	SP-SM Sand
505900	SM Silty Gravelly Sand	SP-SM Sand	SP-SM Sand
5051200	--	SP Sand	SM Gravelly Silty Sand
5051600	SP-SM Sand	SM Silty Sand	ML Sandy Silt
510300	SP Sand	SP Sand	SP Sand
510800	SP-SM Sand	SP-SM Sand	SP Sand
5101300	SM Silty Sand	SP-SM Silty Sand	GP Sandy Gravel

62. Figure 67 presents the monthly lake levels through the monitoring period (December 1982-June 1985). The Great Lakes experienced above-normal supplies (inflows) during this period resulting in levels 1.5 to 3 ft above average. Levels were increasing particularly near the end of the monitoring period and peaked the following year. The increase in water level partially explains the profile changes (i.e., erosion near and above water level with accretion farther offshore). In light of this change, it would be premature to compare the monitoring results with the predicted shoreline presented on Figure 14. Accretion immediately south of the breakwaters and erosion north of the breakwaters were predicted. The profiles south and north of the breakwaters have experienced a general net accretion, primarily occurring offshore.

Channel stability

63. Six sediment samples were collected in Cattaraugus Creek during the spring 1983, fall 1983, and summer 1985 at Sta 55+00, 60+00, 65+00, 70+00, 75+00, and 87+00 with the results presented at Figures 68 through 73. Sounding surveys of the creek were taken December 1982, May 1983, October 1983, July 1984, and June 1985 during the monitoring period. As part of the Operation and Maintenance program, soundings were taken on an annual basis thereafter. Cross sections for Cattaraugus Creek are presented on Figures 74 through 87. Shoaling and movement of the -3.5 ft LWD contour during the period December 1982 to May 1989 are presented in Figure 88. Although streamflow measurements were not taken at the mouth (Drainage Area (DA) = 558 square miles), the maximum, mean, and minimum daily discharges upstream at Gowanda (DA = 436 square miles) for the period are presented in Figure 89. The annual maximum discharge-frequency curve for Cattaraugus Creek at Gowanda is presented in Figure 90. Peak discharges above a base discharge of 8,000 cfs for the monitoring period are presented in Table 13.

64. Sediment observed in the creek during the monitoring period was primarily sandy silt with traces of gravel, although at the mouth in December 1982 a sandy gravel deposit was present. Even though there were no significant streamflow events during the monitoring period, stream discharges were sufficient to move the gravel deposit out into the lake as evidenced by an increase in coarseness of the samples at Sta 501700, 5051200, and 5101300 (Table 12).

65. As discussed in the section on Implemented Harbor Improvements, the channel depths of -5.5 and -3.5 ft LWD initially resulted in a minimum summer navigation depth of 8 and 6 ft of water. It was intended that the bottom elevation would be adjusted based upon long-term lake level changes. From the

Table 13

Peak Discharge Above Base at Gowanda

Date	Discharge, cfs	Percent Frequency
25 Dec 1982	9,160	89
18 Feb 1983	13,500	61
5 Apr 1983	8,080	94
23 May 1983	9,530	87
18 Jun 1983	22,500	16
29 Dec 1983	9,580	86
23 Feb 1984	14,100	57
13 Nov 1984	12,000	71
20 Jan 1985	12,300	71
11 Mar 1985	10,600	81
11 Jun 1985	9,990	84

mouth to Sta 80+00, the channel was stable until 1984 and began accreting 1 to 3 ft until 1989. Upstream of Sta 80+00, the navigation channel has experienced accretion with 2 to 3 ft of deposition occurring from December 1982 to September 1987. The soundings also show that the navigation channel is not optimally located. Since Cattaraugus Creek has a nonlinear alignment, in some areas the deepest part of the stream channel has migrated out of the navigation channel with shoaling occurring within the navigation channel. This condition appears particularly prevalent between Sta 50+00 to 70+00 and Sta 90+00 to 100+00. Based on the shoaling and movement of the -3.5 ft LWD contour presented in Figure 88, two alternative channel alignments (Figures 91 and 92) were developed and are currently under consideration by USAED, Buffalo. Based upon this observation, it is recommended the future channel design accommodate the natural scouring action at the outside of bends to the best advantage.

66. The south breakwater has a filter fabric sediment barrier extending through the structure 850 ft from the landward end. From field observation, it appears that sand has leaked through the structure where the barrier is absent, as seen on Figure 38. However, the growth of this deposit has been

small, which can be attributed to either a low leakage rate and/or sufficient transport downstream by creek discharges.

Ice-jam problem

67. Prior to construction of the project, flooding occurred almost every year along the lower reaches of Cattaraugus Creek. During late winter and early spring, the creek would swell from melting snow and spring rains and cause damage in the Sunset Bay area, the Town of Hanover, and the Town of Brant area in the Cattaraugus Indian Reservation. This flooding was partially due to the limited capacity of the existing creek channel, but the major contributing factor was the presence of a restrictive bar at the creek mouth. This bar, formed mainly by littoral drift, at times virtually closed the outlet and provided a natural barrier encouraging the formation of ice jams. The ice jams caused significantly higher stages and more severe damage than those caused by discharge only. This resulted in considerable flood damage with only moderate creek discharges. Figure 93 presents flood profiles for ice-jam floods of record. Peak discharges at the Gowanda gaging station for these floods are presented in Table 14.

Table 14

Peak Discharges at Gowanda During Ice Jams at Mouth

Date	Peak Discharge at Gowanda, cfs
19 Feb 1961	*
17 Mar 1963	17,100
2 Mar 1972	14,000
17 Feb 1976	19,000
23 Feb 1985	14,100

* Not reported, but less than 15,000-cfs base discharge and greater than daily discharge of 5,800 cfs.

68. Qualitative tests were conducted during the hydraulic model study to determine any ice-jamming tendencies, as discussed earlier. Ice flow tests revealed no ice-jamming tendencies between the breakwaters for various creek discharges. At the time of the hydraulic model study, expertise in ice modeling was beginning to emerge. The use of polyethelyene chips to model ice blocks was the state of the art and the most anyone could do given the modeling facilities available. The ability to simulate a solid lake/river cover was not possible. The ice near the mouth was simulated by constructing

a jam between the breakwaters (Figure 13) and subjecting it to various discharges. As the jam broke at a low discharge of 10,000 cfs, it was determined qualitatively that ice jamming was not a problem. However, since the project was completed in January 1983, several ice-jam flood events have occurred (USAED, Buffalo 1987). During the winter of 1983-84, minor street flooding occurred because of ice jamming, but because damages were relatively insignificant, the event was ignored. During the period 23-25 February 1985, a significant flood occurred with a flood stage approximating the 17 February 1976 ice-jam flood of record at the index point. The shape of the profiles differed markedly from the February 1976 flood as shown on Figure 93. The February 1976 flood had a peak discharge of approximately 19,000 cfs (3.6-year recurrence interval) at the Gowanda gage, and the 23 February 1985 flood had a little lower discharge of 14,100 cfs (1.8-year recurrence interval).

69. During the period 19-21 January 1986, another ice-jam flood occurred at Cattaraugus Creek Harbor. Data for this event indicate that the scale discharge at Gowanda was 12,300 cfs and the maximum level at Sunset Bay was almost equal to the February 1985 event. During the ensuing 2 weeks, bitter cold temperatures resulted in significant ice growth. Another warming trend on 4-5 February 1986 resulted in a peak discharge at Gowanda of 6,700 cfs and another ice jam, but with only minor overbank flooding. This was due largely to the efforts of ice-breaking equipment that partially broke up the harbor ice cover prior to ice runoff and the low creek discharge.

70. On 2 March 1987, about 200 residents of Sunset Bay were evacuated when ice again jammed the creek near the mouth and runoff waters threatened to inundate homes. The peak discharge for this event at Gowanda was 5,700 cfs. Although the creek exceeded its banks, only minor flooding occurred, and no significant damage resulted. A higher peak discharge of 7,200 cfs occurred 6 days later with no reports of damage.

71. The occurrence of two major ice jams within 4 years after project completion resulted in questioning whether the project design was deficient (USAED, Buffalo 1987). It was determined that prior to project construction, ice would jam at a minimum of two locations near the mouth. Initially it would occur upstream of the mouth at the New York Central Railroad Bridge. As the water behind the jam rose, the pressure would finally break the jam, and the flow would progress downstream to the mouth (USAED, Buffalo 1963). At the mouth, a second jam formed when the ice arrested at the sandbar. The sandbar normally extended 1 to 2 ft above the lake surface and was covered by

windrowed lake ice in the winter. As the channel filled, water overflowed the banks just below the railroad bridge. On the right bank, the water and ice traveled overbank to an old abandoned channel of Cattaraugus Creek. The velocity and strength of the water opened this abandoned channel to the lake and provided an outlet for part of the floodwater. On the left bank, the floodwater and ice passed over the low banks downstream of the railroad embankment and passed through Sunset Bay to the lake. The flood found two avenues at the lake, one through the area known as Parson's Pond and the other at a relatively low section of the beachfront near Iola Drive. Figure 94 presents flooded outlines for the 1961 and 1963 ice-jam floods.

72. It is the opinion of CRREL personnel that the breakwaters have little or no influence on the initiation or location of the ice jams (Zufelt and Den Hartog 1985). However, the north breakwater berm prevents water from spilling over the beach on the north side, which occurred during preproject conditions and thus raises ice-jam flood levels (USAED Buffalo 1987; Zufelt and Den Hartog 1985). During construction of the berm, additional stone was required because a recent flood had scoured that area. Figures 95 and 96 present conditions after the February 1985 ice-jam event. Note the large quantity of ice present in the right overbank (north) in Figure 96.

73. The primary reason for the occurrence of ice jams at the mouth of Cattaraugus Creek is the presence of lake and river ice cover. The ice run dissipates its energy by attempting to break the fixed ice sheet. If it is unsuccessful, a jam occurs. The severity of the resulting flood is a function of the thickness and extent of the ice sheet, ice run quantity, and stream discharge. With a minimum ice sheet or if the integrity of the sheet is broken by human intervention, the river ice continues to move into the lake. During the 4-5 February 1986 floods, only minor overbank flooding occurred largely because of the efforts of ice-breaking equipment that partially broke up the harbor ice cover prior to the ice runoff.

74. In addition, it is hypothesized that the presence of the shore protuberance on the right bank near Sta 63+00 causes a restriction that may retard the ice passage. It may also deflect some of the ice run to the south breakwater, where it may be retarded as it rubs along the structure. A small shoal builds along the channelside toe of the structure, lakeward of the end of the filter cloth from sand transmission. It is transient as it is scoured by creek flows. It was not present during the February 1985 and January 1986 flood, as evidenced by the channel sections at Sta 57+00, but may retard the

ice passage by a small amount. The total effect of these shoals is unknown, and they are considered secondary to the main cause, that is, the presence of a lake/river ice cover.

75. To alleviate the increased flood damages estimated at $87,800 on an annual basis (USAED, Buffalo 1987), a rectification project consisting of lowering the north breakwater berm from +12.0 to +5.5 ft LWD was proposed. A 400-ft length of the berm was proposed to be lowered with construction scheduled for the summer of 1987, provided that a right-of-entry be obtained from the Seneca Nation of Indians. It was not obtained, and there is little potential for implementation in the foreseeable future. Figure 97 presents the proposed modified berm cross section.

76. The reduction in the berm crest height was estimated to reduce average annual damages by 50 percent but would not prevent ice-jam formation. Possible solutions were identified during the Cattaraugus Creek Study (USAED, Buffalo 1987) and evaluated in a two-stage iterative process. Nine preliminary alternatives in addition to the "no-action" option were initially formulated and assessed. These alternatives fell into two broad categories: local protection plans in areas where a high concentration of flood damages exists (Sunset Bay and Arcade) and dam reservoir plans at Springville. For the plans at Springville, hydroelectric power generating facilities and recreation facilities were also considered to maximize the economic efficiency of the basic flood-control plans.

77. Two promising plans were the construction of an overflow channel in the right overbank near the mouth and the construction of an ice retention structure upstream of the town of Versailles. Further investigation of the overflow channel showed it to be not economically feasible. The plan would have consisted of a 4,600-ft-long overflow channel, 400 ft wide, and 2 to 3 ft in depth. A protective berm would be provided adjacent to the left bank of the channel to prevent the cottages in Snow's Marina from being flooded when the channel is carrying flood flows. The access road would be lowered where it crosses the channel, and the beach area at the downstream end of the overflow channel would also be lowered to +5.0 ft LWD. Maintenance dredging at the mouth of the channel and annual loosening and breaking the windrowed ice that forms along the lakeward margin of the beach berm would be performed. Beach profile data gathered under MCCP authority were used to evaluate the maintenance dredging requirement. This plan is presented in Figure 98.

78. The selected plan consists of a 270-ft-long ice retention structure, a 200-ft-wide adjacent floodway for passage of flood flows, a fish ladder with sea lamprey control barrier, and fisherman access facilities just upstream of the town of Versailles. The ice retention structure would have three gated low-flow openings to accommodate summertime flows. The operation plan would include closing the gated low-flow openings during the late fall and early winter ice-forming period for the purpose of forming a stable ice cover. The pool (and stable ice cover) would be maintained during the winter and would prevent ice from flowing down the creek and jamming at the creek mouth. The pool would be drained in the spring when the threat of ice-jam flooding was over. Although economically feasible (B/C = 1.56), there has been no local support for this plan. This plan is presented on Figure 99.

Observations

79. As indicated previously, it was observed that a 4-ft scour hole had developed off the head section of the south breakwater some time during the period August 1984 to June 1985. During maintenance soundings of the entrance channel in June 1989, additional soundings in the vicinity of the June 1985 scour hole were obtained. These data revealed that the scour hole had generally filled in except for a very small portion approximately 650 ft from the structure head. Minor offshore steepening had occurred, and a shoal had developed at the mouth of the entrance channel. Figure 100 presents the offshore bathymetric changes in the vicinity of the south breakwater head.

80. Stone locations at Sta W-5 and W-6 were also obtained at this time, as well as structure cross sections. It seems that stone movement at the south breakwater head is continuing, and the structure slope is steepening. It appears that the sacrificial toe berm has probably settled, which may have been the result of the earlier scour, but its presence helped reduce the amount of potential damage. Table 15 presents the change in stone location, and Figures 101-105 present a comparison of the June 1985 and June 1989 cross sections of the south breakwater head.

81. For comparative purposes, Figures 106 and 107 present vertical aerial views of the south breakwater head section taken 16 May 1984 and 3 June 1986, at a scale of 1 in. = 20 ft. Adjacent to the end of the concrete walkway (circular target) is the navigation light's concrete foundation. Inspection

Table 15

Change in Stone Location, South Breakwater Head

June 1985 to June 1989

Line	Point	June 1985 to June 1989		
		Δr, ft	Δθ, deg	Δz, ft
W5 to A	W5A	0.12	0.40	-0.2
W5 to B	W5B	-0.90	10.57	+0.5
W5 to C	W5C	0.12	0.0	-0.3
W5 to D	W5D	-0.02	0.01	-0.5
W6 to A	W6A	-0.23	-6.87	+1.3
W6 to B	W6B	--	--	--
W6 to C	W6C	2.23	2.46	+0.3
W6 to D	W6D	--	--	--
W6 to E	W6E	-0.01	-0.04	+1.0
W6 to F	W6F	--	--	--
W6 to G	W6G	3.93	-22.17	+2.6
W6 to H	WGH	--	--	--
W6 to I	WGI	2.31	-7.28	+1.5
W6 to J	WGJ	5.4	-2.02	-1.2

of the photograph reveals that the rectangular-shaped stone along the breakwater coastline adjacent to the navigation light foundation moved slightly lakeward as evidenced by the gap between them. This would indicate that other stones lakeward of this one had moved lakeward during this period. The third stone (rectangular shaped) located approximately along the center line from the navigation light seen in the 1984 photograph is not visible in 1986. This may be partially due to the slightly higher water level (1.3 ft). However, it may also be due to a general outward movement with a reduction in elevation of the stones lakeward of the light.

82. On 21 September 1989, a site visit was made by a USAED, Buffalo, team to perform a dive inspection of the south breakwater head. The dive was not performed because of poor visibility created by a sediment plume from the creek mouth driven around the head by an east wind; nevertheless, a cracked stone count was made. Armor stones were observed for possible cracking in a 180-deg segment centered on the south breakwater light standard. Only the outer layer of armor stones, primarily above the waterline, was checked. Of 48 stones checked, 16 cracked stones (33 percent) were observed, of which 7 were fragmented. This is a significant number of cracked stones.

83. The shattering of armor stones makes them more vulnerable to movement by gravity and wave forces. As the smaller fragments are dislodged from their original position and move down the slope, adjacent stones collapse into the resultant voids. The damage is expected to continue, and USAED, Buffalo, intends to repair this section. A cursory investigation of the condition of the stones along the trunk section of the south breakwater revealed that some stones had cracked but, in general, the trunk was in good condition.

PART III: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

84. Although waves experienced at the project site have been lower than the design wave, localized damage has occurred at the south breakwater head. The primary cause of the damage appears to be stone cracking. The loss of a few stones by shattering causes adjacent stones to collapse into the void, resulting in a steepening of the structure slope. The problem has been recently recognized by USAED, Buffalo, as causing damage at other structures. At Cleveland Harbor, Ohio, it was found that the majority of cracked stones were located at or above the waterline.

85. Very little stone movement occurred during the monitoring period, as might be expected since the design conditions were not experienced. Armor on the lake side of the south breakwater moved perpendicular to the breakwater center line -0.11 to +0.15 ft with an average movement inward (landward) of +0.01 ft. The stones rotated about the target -1.33 to +0.2 deg for an average clockwise movement of 0.23 deg. Vertical stone movement was -0.1 to +0.2 ft with an average upheaval of 0.05 ft. Ice had negligible effect upon the structure except for minor damage to one hand rail at the south breakwater head, indicating that stone sizes based on the local wave climate were sufficient for ice conditions as well.

86. Precautions applied during design to protect the structure toe have had the desired effect. Based on experience with structures located on erodible material, USAED, Buffalo, often used additional toe protection, a technique that has repeatedly produced a stable toe.

87. Sediment transport patterns were predicted reasonably well during the design process, but the magnitude of the transport was underestimated. Predicted transport was 35,000 tons/year (23,350 yd^3), while the calculated average transport over the entire area was 95,700 cubic yards/year.

88. It was found that the sediment generally got coarser only offshore of the creek entrance because of improved transport of coarse material out of the entrance by the streamflows. Another indication of the improved flows was the appearance of debris on the beach north of the creek mouth. The coarse material and debris were no longer being trapped on a bar at the creek mouth.

89. The shoreline to the south of the structures has responded as expected. Erosion immediately to the north has not occurred. Instead, there

has been accretion for a considerable distance to the north, much farther than anticipated.

90. The scour hole that appeared off the south breakwater head was probably due to local wave effects and increased currents near the head. The reason it filled in is not completely understood, but may be as a result of the natural bypassing of material around the south breakwater as the fillet grew combined with the transport associated with lower lake levels.

91. While the maintenance dredging had been anticipated, it has not been required until now. The need for dredging now exists more as a result of channel migration near the mouth than of shoaling. With regard to dredging, the project has performed well, although a modification in the channel alignment is being considered. This MCCP study was the catalyst for this recommendation.

92. The use of filter fabric was successful, at least to date. There were obvious indications of transport through the south breakwater lakeward of where the fabric had been installed (Figures 50 and 51). Figure 50 shows conditions with waves entering the mouth of the creek and pushing more sediment-laden water through the structure. Figure 51 shows the opposite conditions, with the clearer lake water penetrating into the channel. These are indications that sediment has been prevented from penetrating the structure and reaching the channel where filter fabric has been used.

93. As might be expected, there was deposition on the inside of bends in the channel and scour on the outside of those bends.

94. The inability to model lake ice prevented the reproduction of flooding when lake ice stopped ice flows from the stream. This did not allow the problem associated with the north berm to be modeled.

95. The use of ice-breaking equipment to break up harbor ice helped prevent flooding. Buffalo's ice breaker vessel was used for this purpose.

96. The physical model did an excellent job in identifying the best way to eliminate shoaling in the navigation channel, preventing ice jams, recognizing the limitations of the state of the art in modeling lake ice, and designing a channel safe for navigation in high wave conditions.

Recommendations

97. There needs to be further investigation to identify the cause of stone cracking so the problem, which is becoming significant for structures on

the Great Lakes, might be avoided in the future through better material specifications.

98. Experience in the Great Lakes appears to justify the use of 0.9 to 2.0 W stone weight range in design rather than that called for by the Shore Protection Manual (1984). It would be useful to further investigate the performance of structures using both stone weight criteria to identify which provides the most cost-effective design.

99. Use of toe protection in this case where the lake bottom is susceptible to erosion has prevented structure failure and should be used in all similar cases.

100. When selecting the method for evaluating sediment transport, the designer must be careful. While none appear perfect for any situation, some appear much less sophisticated than others. The selection of a technique must be tempered with experience and specific existing conditions.

101. During the design of a project of this type, it is important to remember all the results of improved flows. Neither the increased transport of coarser material or the increase in debris on the north shore has been a problem, but it is important to be able to predict all the effects. For example, not long after construction of the project, coarser material appeared on the state beach a few miles to the north of the project site.

102. Localized effects such as wave refraction and diffraction near the structures must be considered when performing the design. These can offset potential sediment losses near these structures.

103. The potential for scour near the head of coastal structures, and the appearance of such a scour hole at Cattaraugus Creek, is additional justification for adequate toe protection. In this case, it did not have adverse effects, although additional toe protection in the form of a small stone or gravel mat would have prevented the hole from developing. It should be mentioned that a much deeper scour hole recently developed at the head of the Irondequoit west breakwater in Lake Ontario and required filling with large stone. The prevalence of these phenomena, the factors which cause their development, and recommended design criteria should be investigated.

104. The breakwaters at Cattaraugus Creek have performed admirably. This design solution should be considered at locations experiencing similar problems.

105. Recognizing placement limitations, such as where large waves are anticipated, and an unknown life expectancy, the use of filter fabric to help

make a structure impermeable may be a good idea. So far it has prevented sediment transport through the structure at a relatively small cost. Periodic inspections should note its continued performance.

106. The natural relocation of the channel at Cattaraugus Creek is further evidence that consideration should be given to accommodating natural scour at the outside of bends when designing a channel alignment. It is possible that dredging requirement can be reduced.

107. Experience at Cattaraugus Creek supports the lowering of the berm on the north side of the creek entrance, as has been recommended (USAED, Buffalo 1987), if permission can be obtained from the Seneca Nation.

108. As a part of the design process, consideration should be given to whether, and under what conditions, ice-breaking equipment could be used to advantage.

109. The physical model used in evaluating design alternatives has proven to be an excellent tool. Efforts should continue to improve the capability to model lake ice. This capability would increase the value of physical models where ice conditions must be considered.

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Figure 1. Aerial view of Cattaraugus Creek Harbor, New York (May 1983)

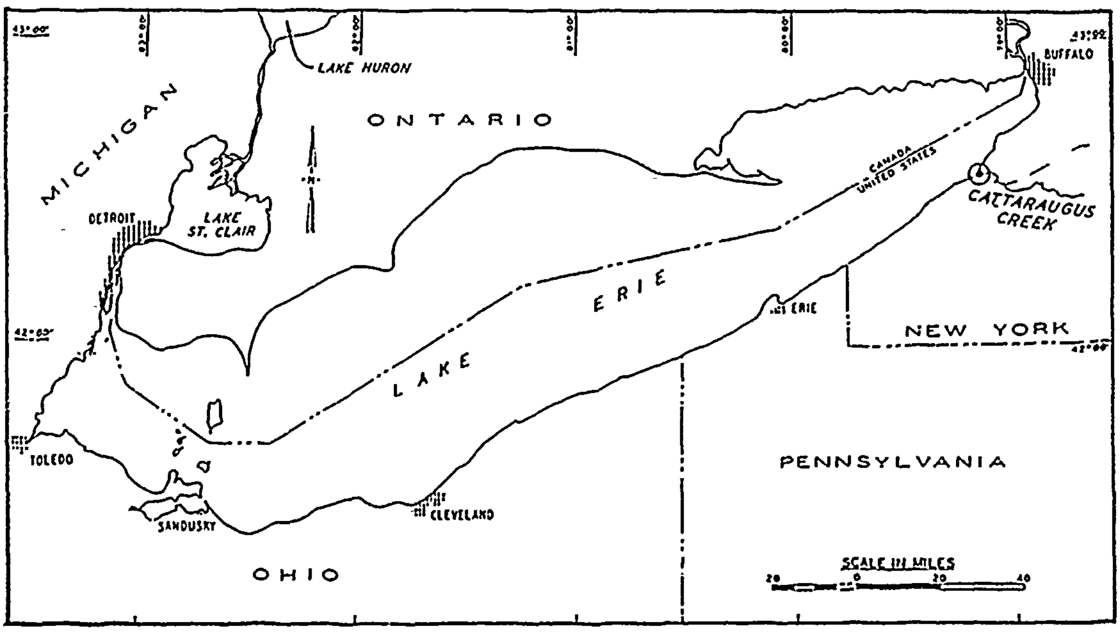


Figure 2. Location of Cattaraugus Creek

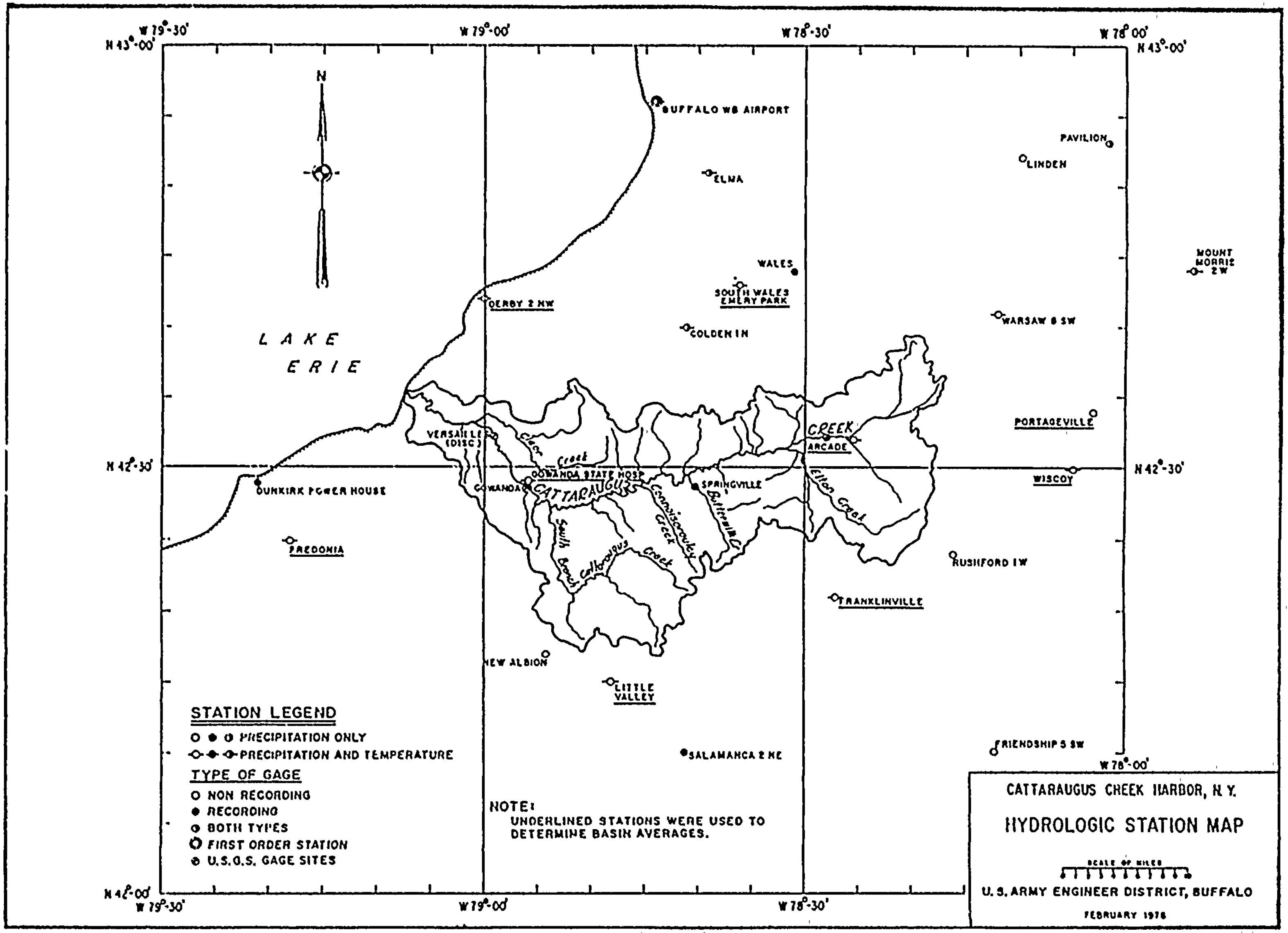


Figure 3. Watershed and hydrologic station map

Figure 4. Cattaraugus Creek mouth, June 1974

Figure 5. Cattaraugus Creek mouth, September 1980

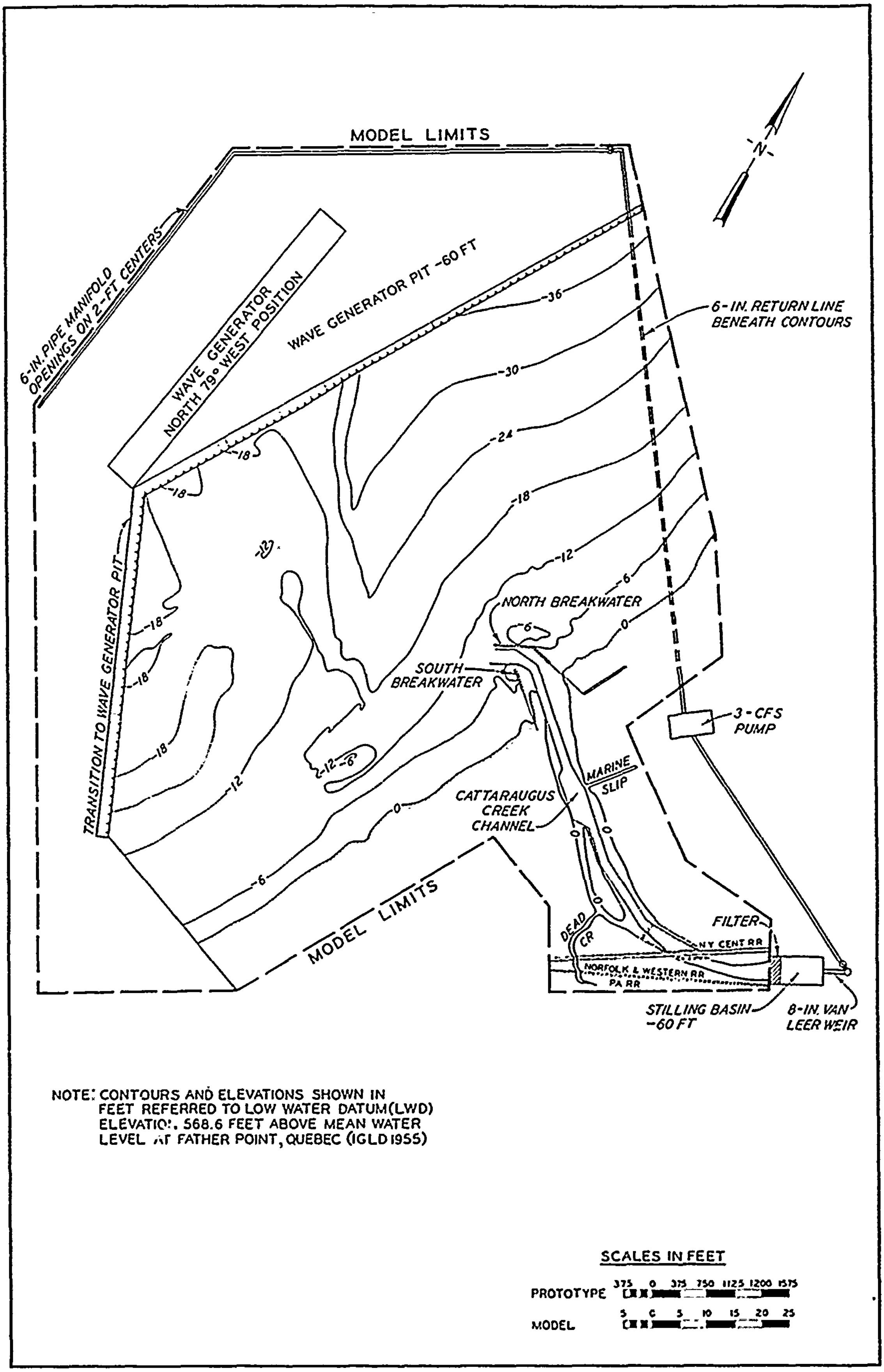


Figure 6. Model layout

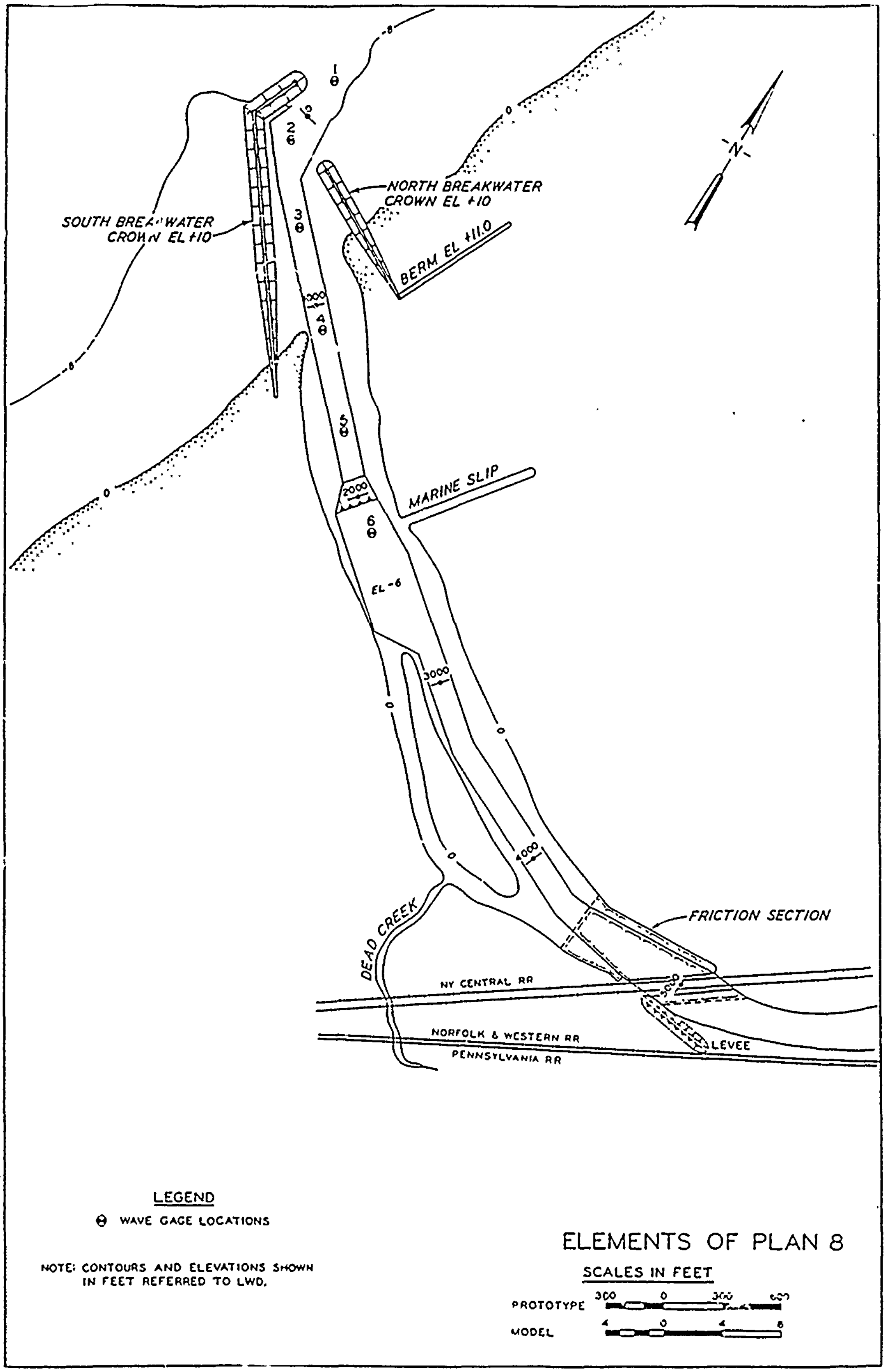


Figure 7. Hydraulic model, Plan 8

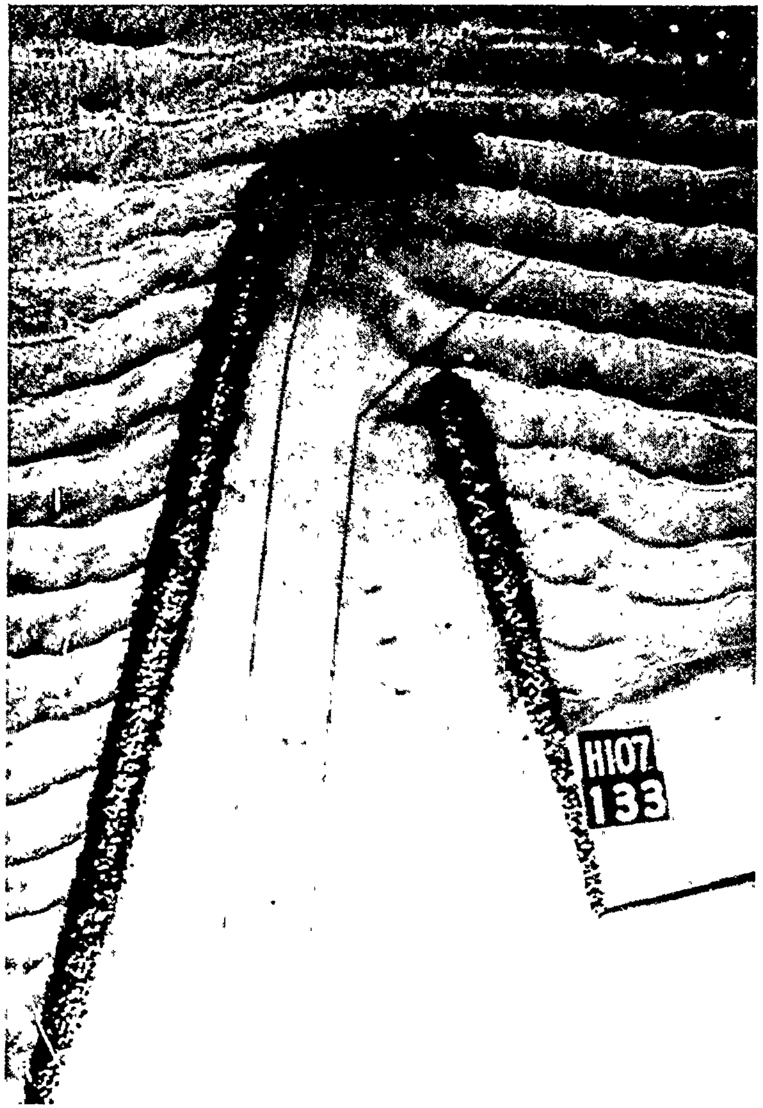


Figure 8. Model study, Plan 8, 6-sec, 9-ft waves from northwest

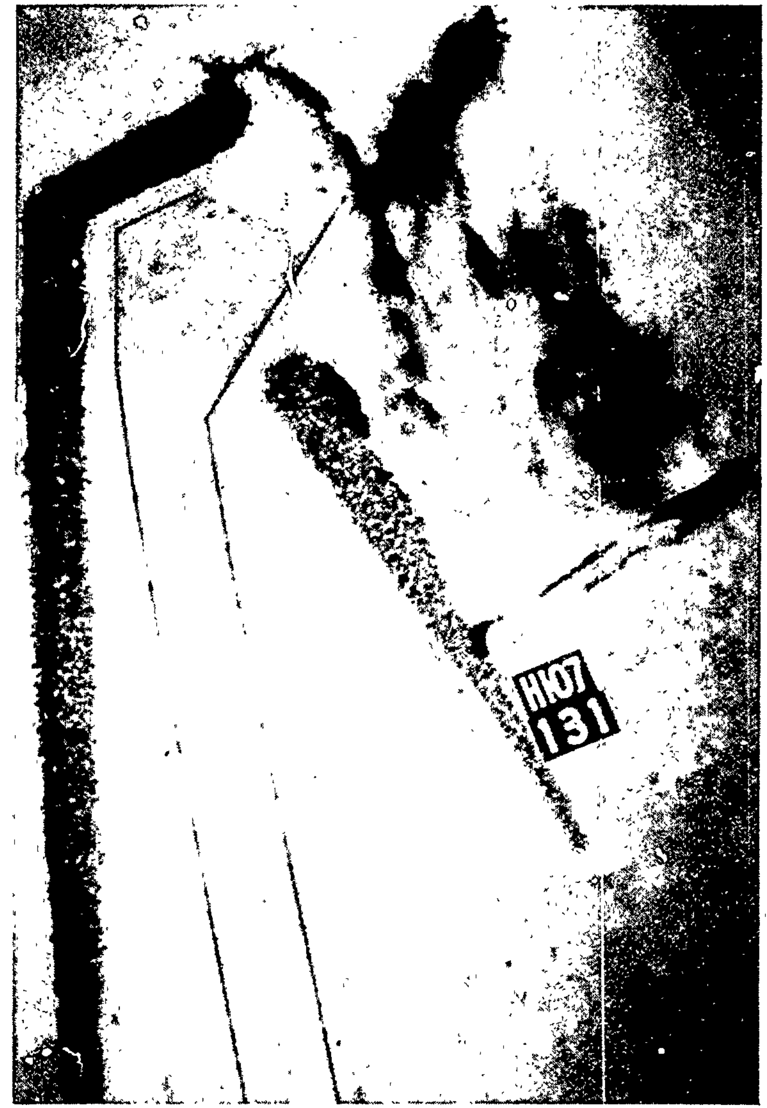


Figure 9. Model study, Plan 8, tracer deposits, 6-sec, 9-ft waves from northwest

Figure 10. Model study, ice location prior to creek discharge for base conditions

Figure 11. Model study, ice movement with discharge of 10,000-cfs for base conditions

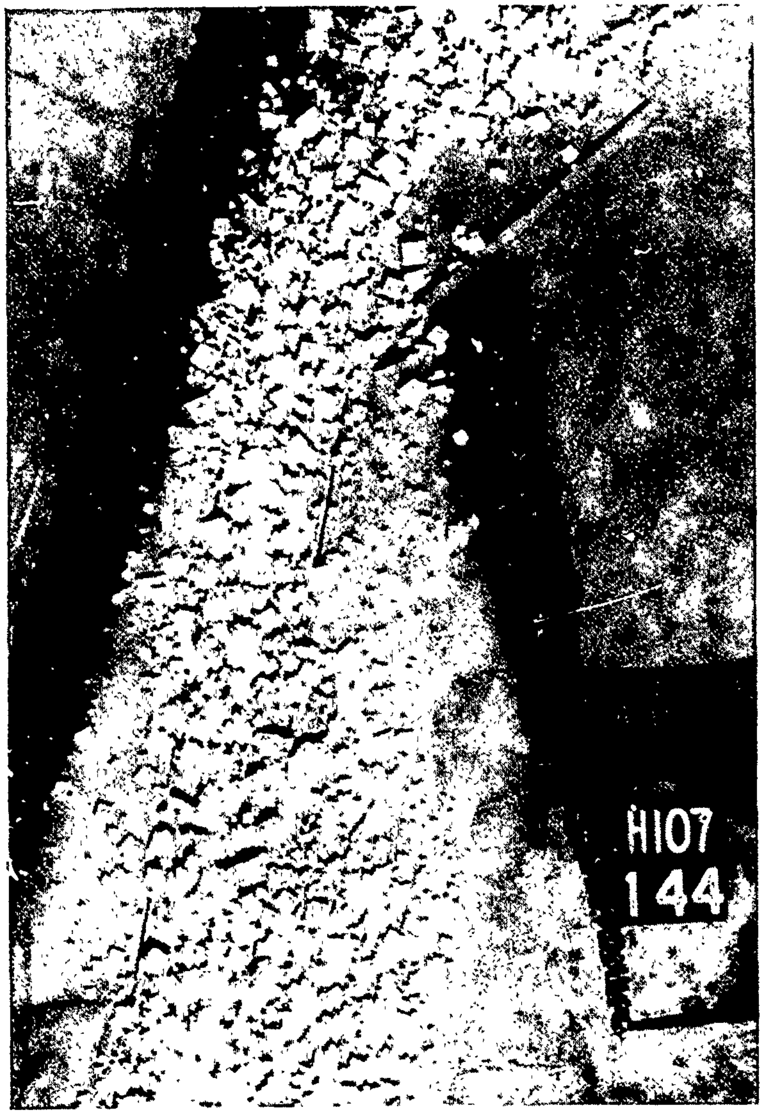


Figure 12. Model study, Plan 8, ice progression with 5,000-cfs discharge

Figure 13. Model study, Plan 8, ice jam constructed between breakwaters

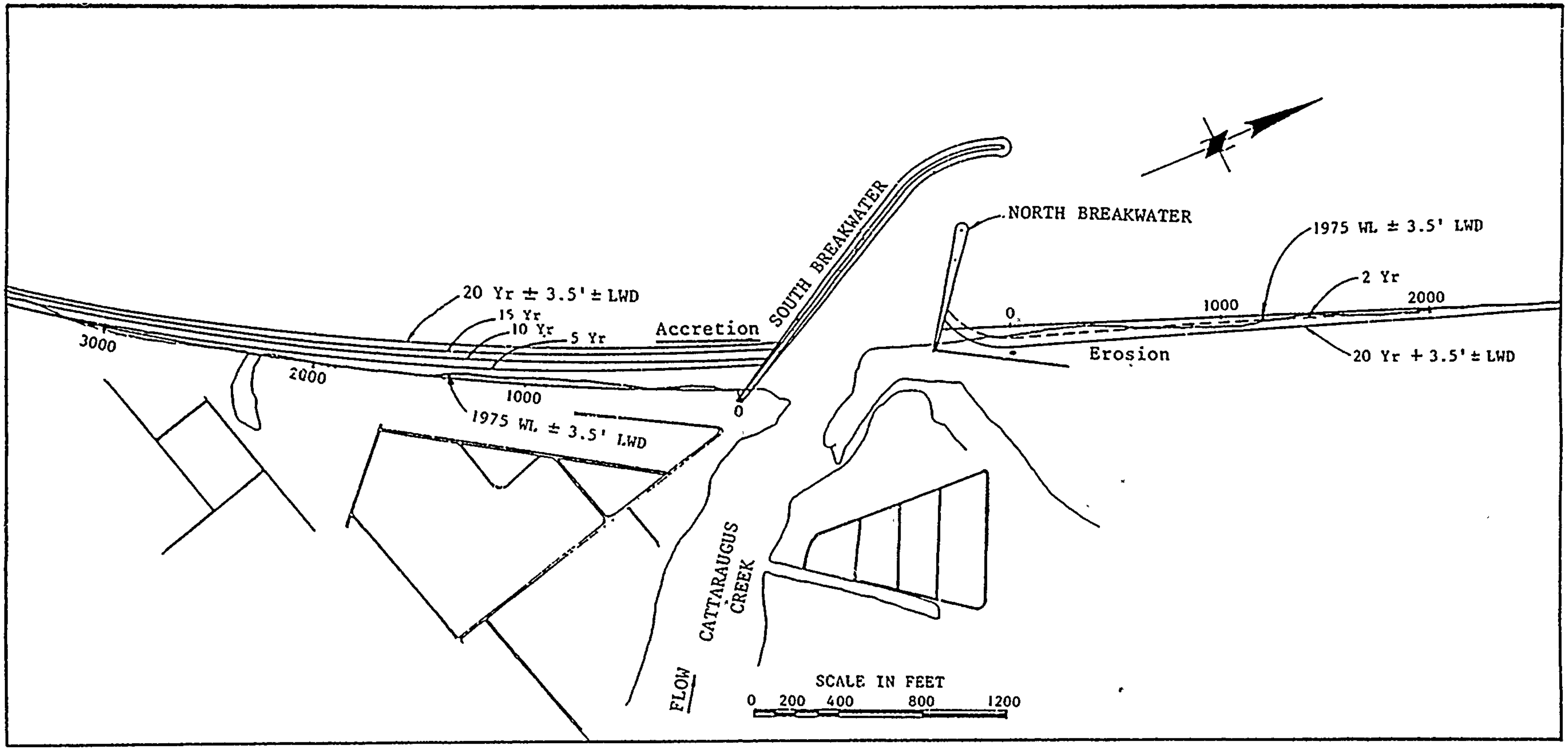


Figure 14. Predicted shoreline accretion and erosion

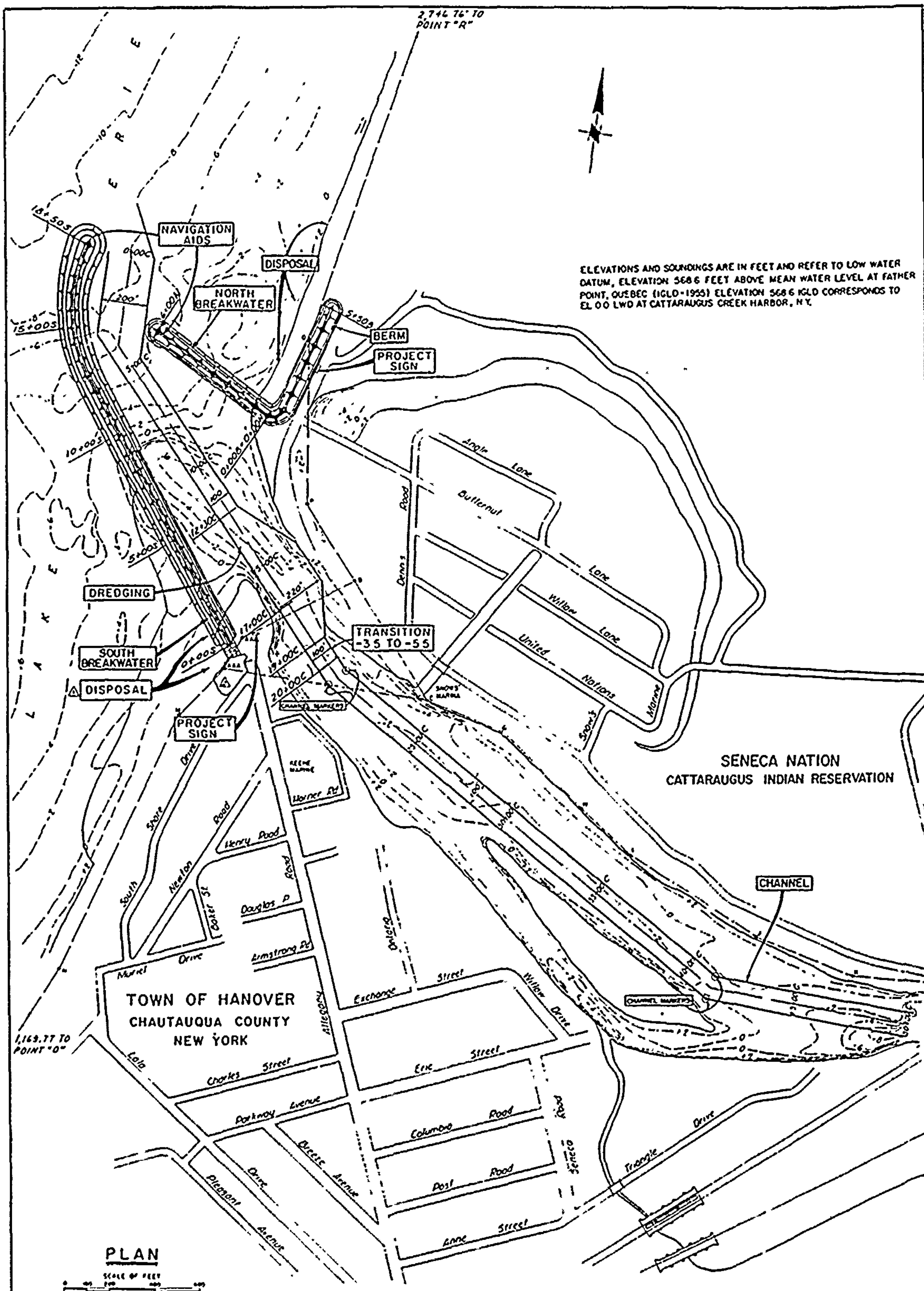
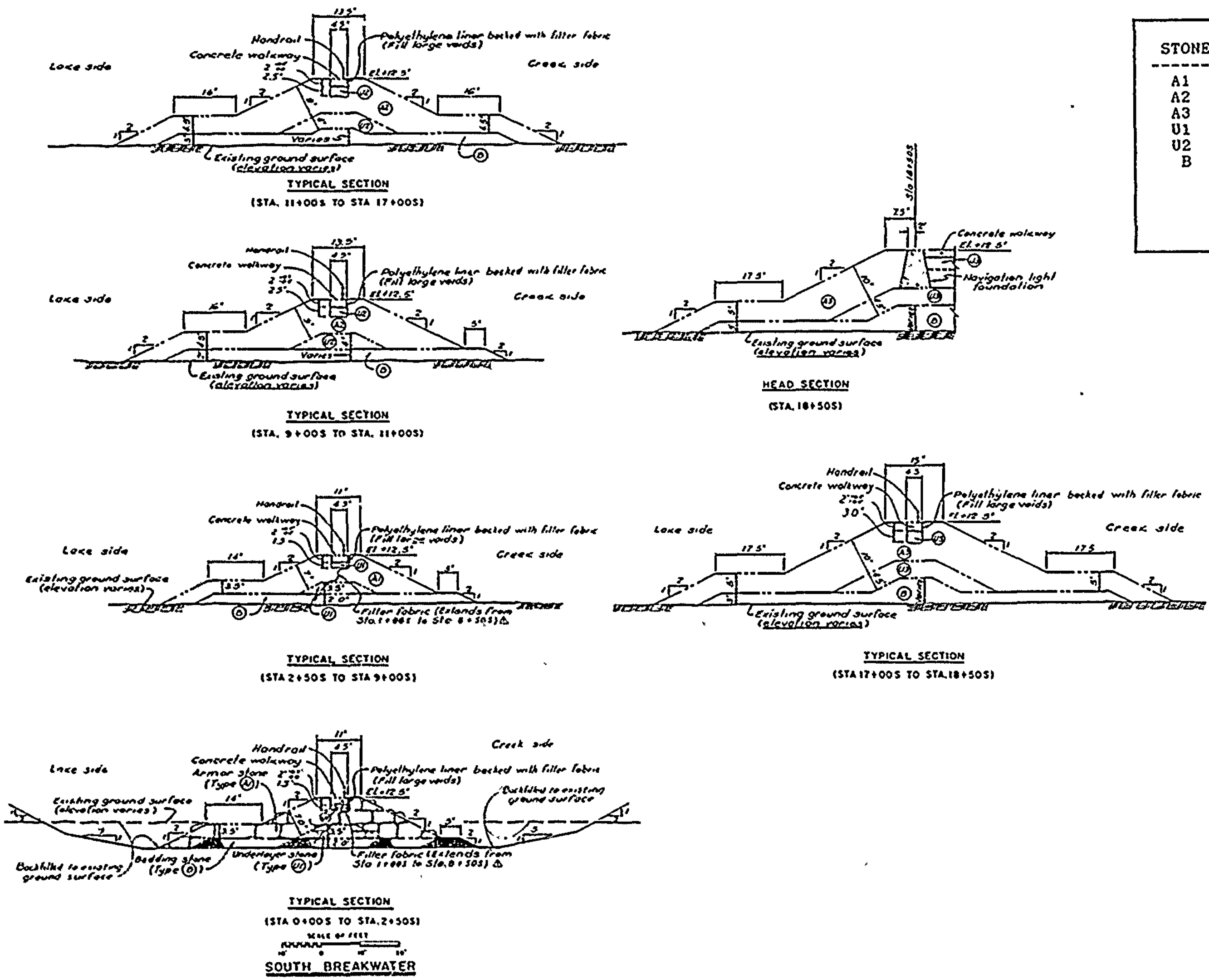


Figure 15. Project, as constructed, at Cattaraugus Creek



STONE	SIZE RANGE
A1	2.0 - 5.0 TONS
A2	4.0 - 9.0 TONS
A3	6.0 - 13.0 TONS
U1	240 - 950 POUNDS
U2	460 - 1850 POUNDS
B	10" - 100%
	6" - 35 TO 100%
	3" - 0 TO 40%
	1" - 0 TO 10%
	#16 SIEVE - 0 TO 5%

Figure 16. Construction drawings of South Breakwater cross sections

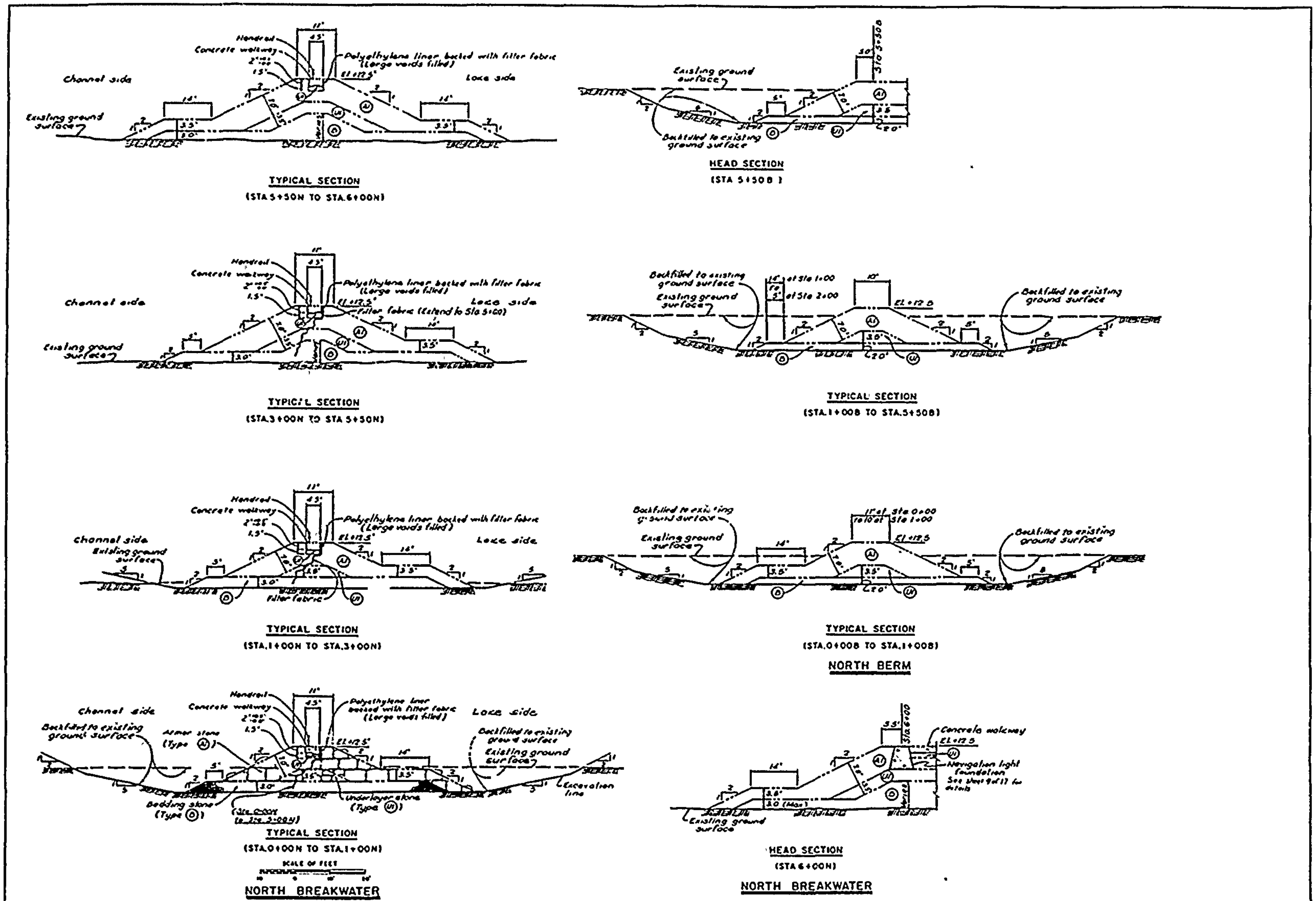


Figure 17. Construction drawings of North Breakwater cross sections

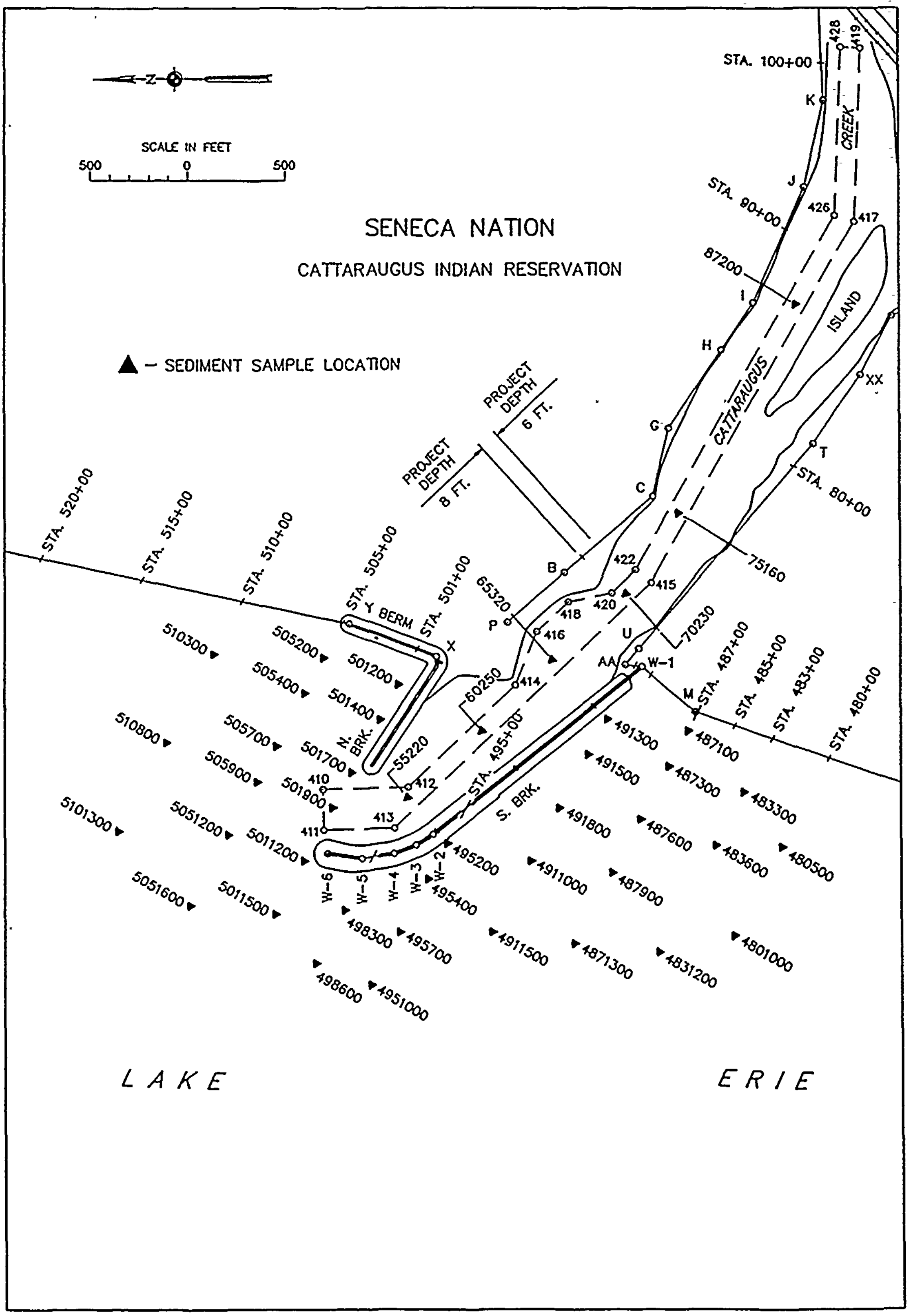


Figure 18. Stations and sediment sample locations

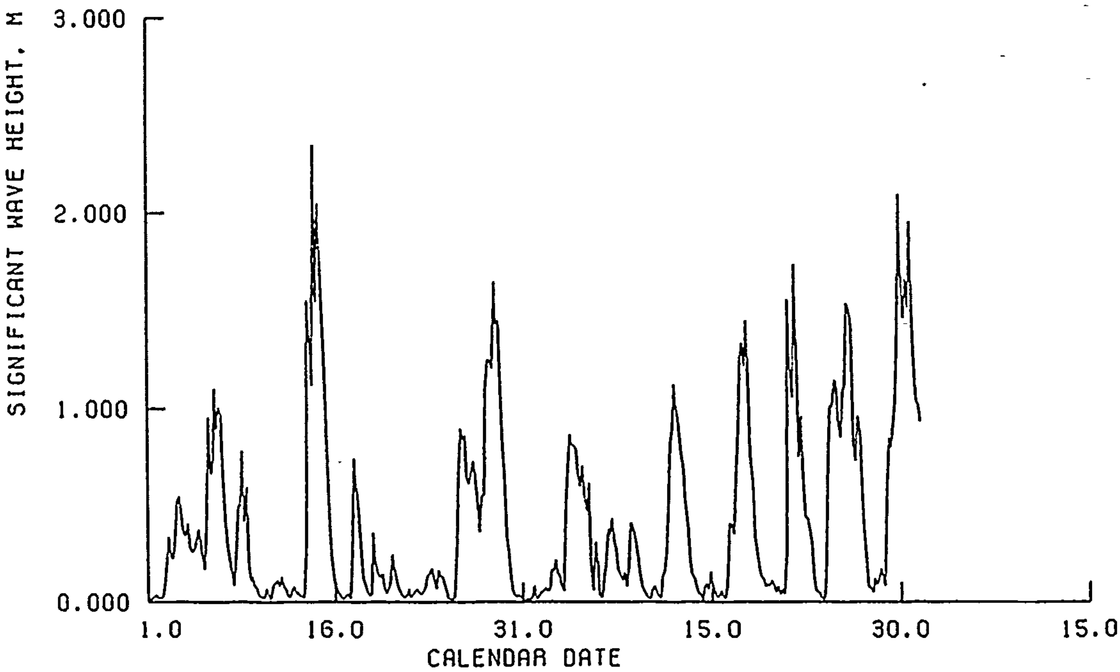


Figure 19. Significant wave height, October-December 1983

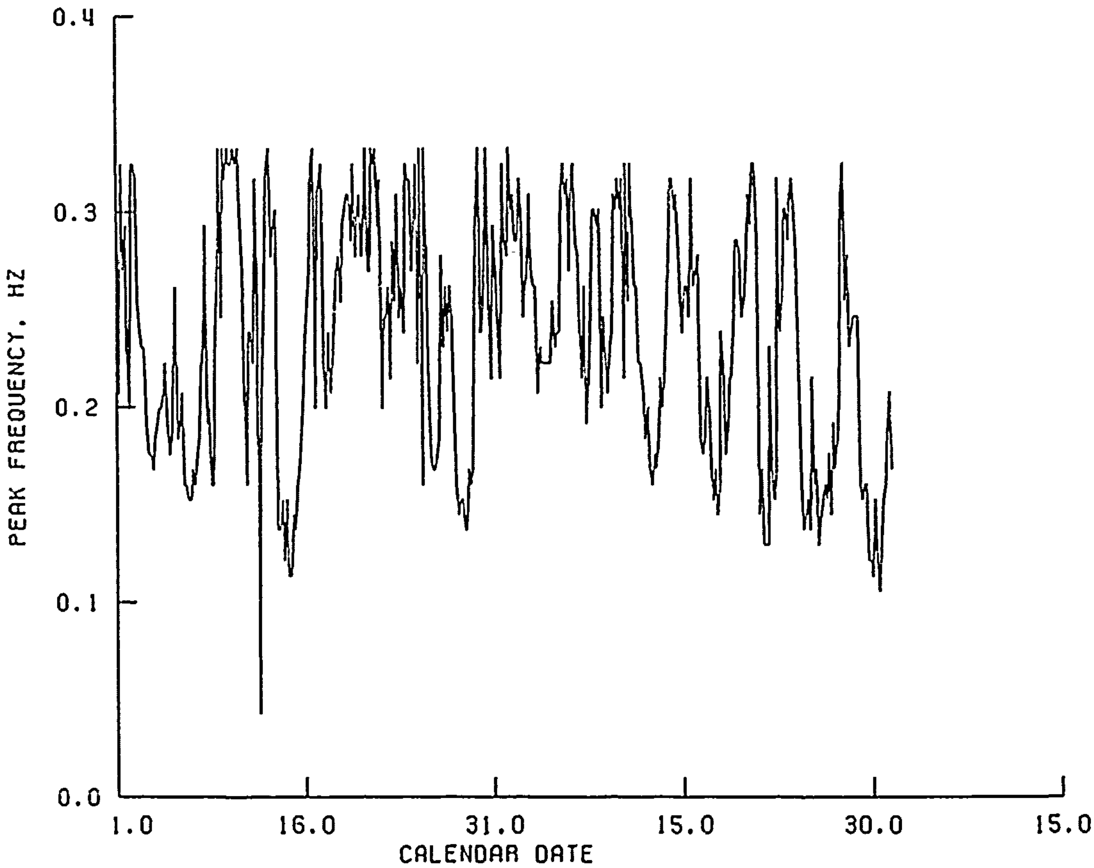


Figure 20. Peak frequency, October-December 1983

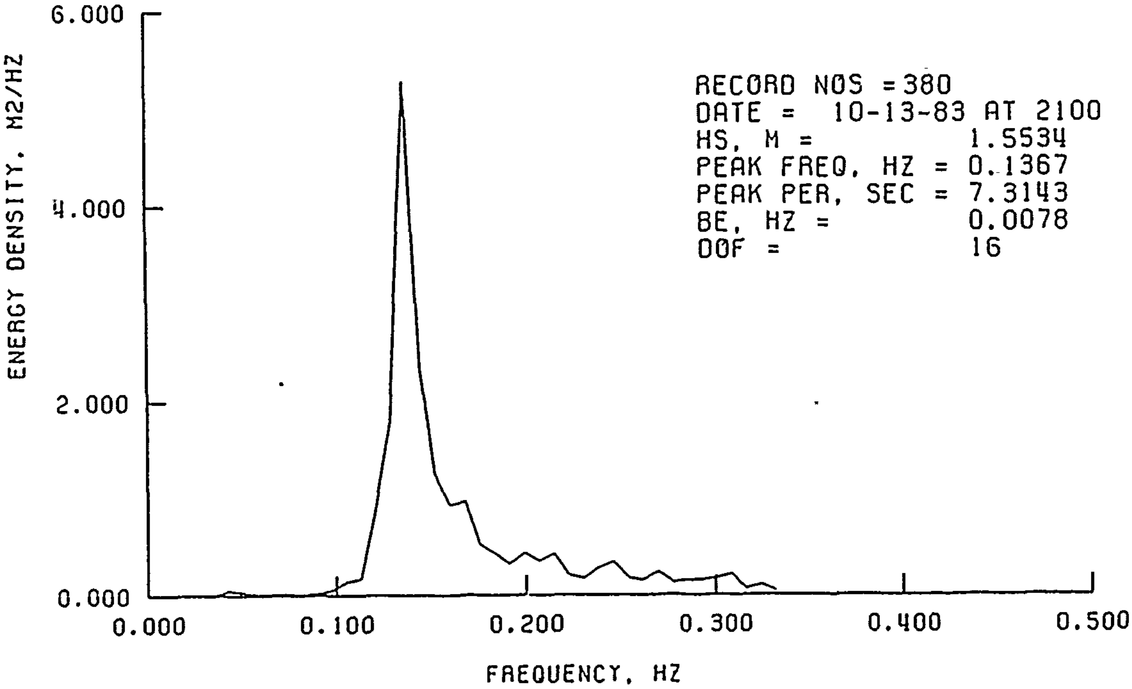


Figure 21. Wave spectra, 2100 hr, 13 October 1983

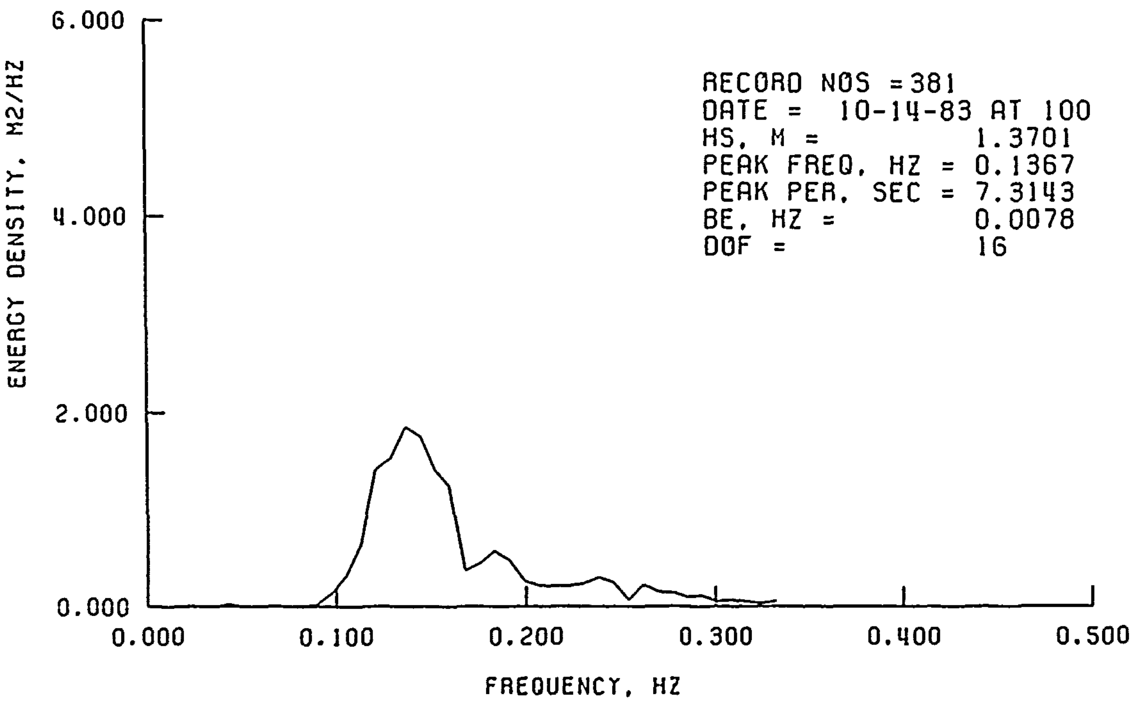


Figure 22. Wave spectra, 0100 hr, 14 October 1983

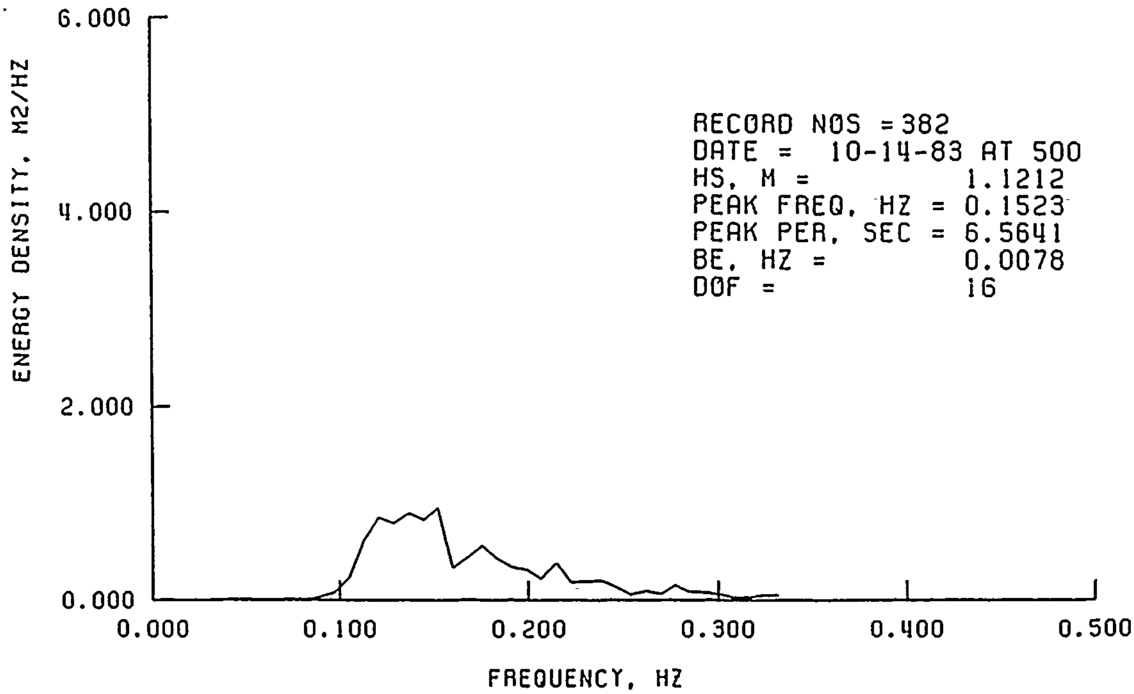


Figure 23. Wave spectra, 0500 hr, 14 October 1983

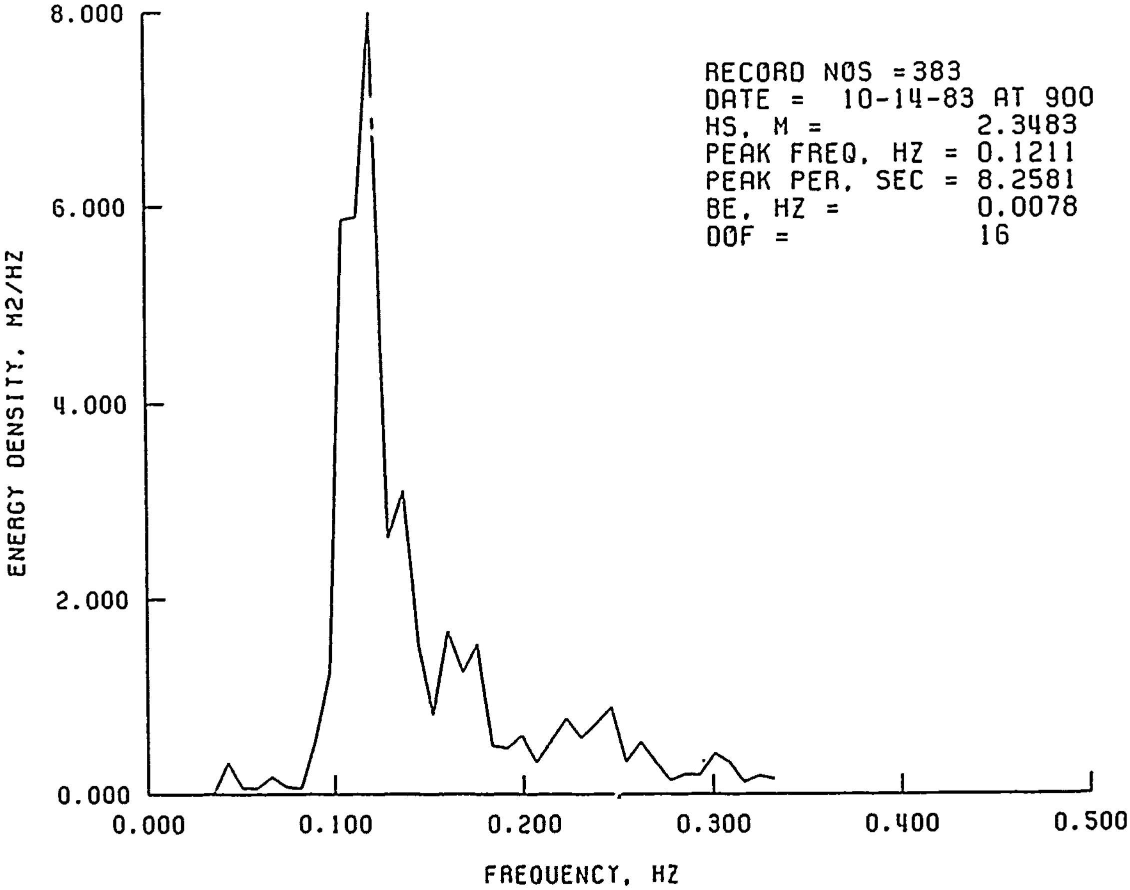


Figure 24. Wave spectra, 0900 hr, 14 October 1983

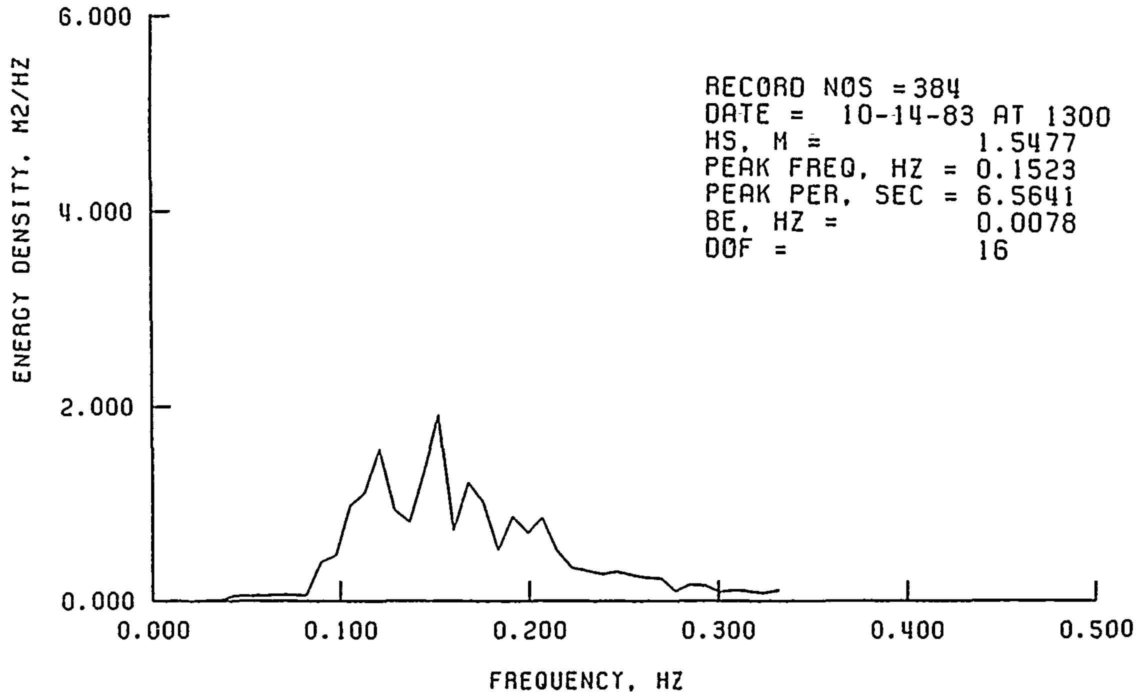


Figure 25. Wave spectra, 1300 hr, 14 October 1983

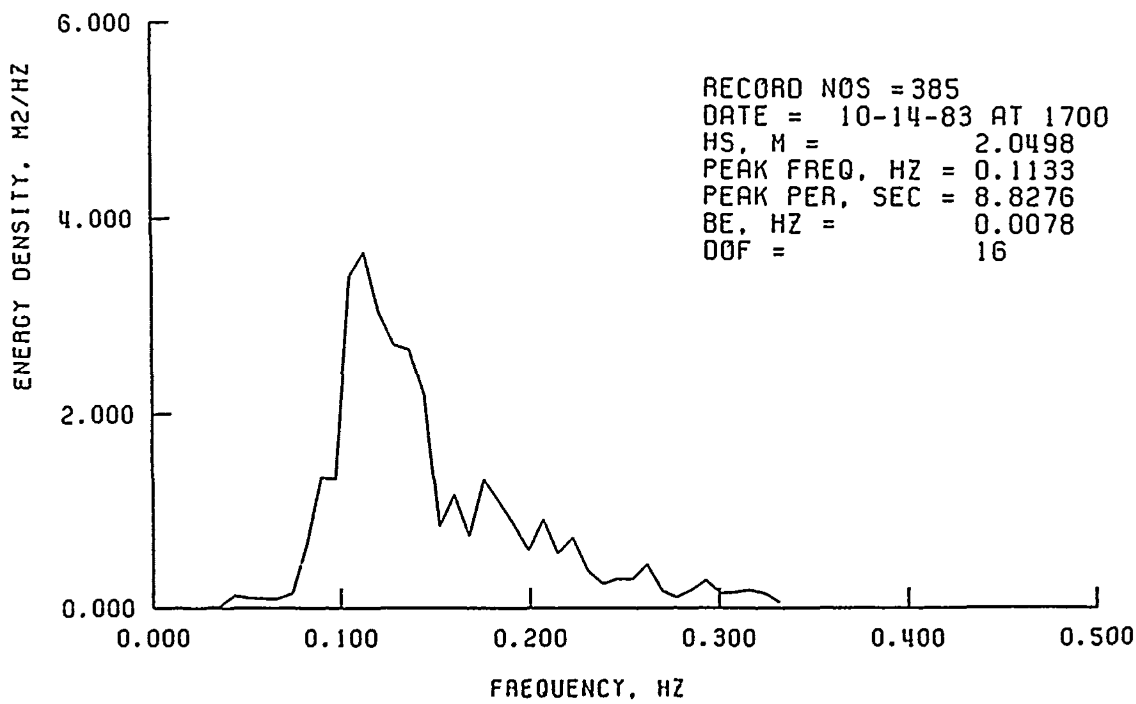


Figure 26. Wave spectra, 1700 hr, 14 October 1983

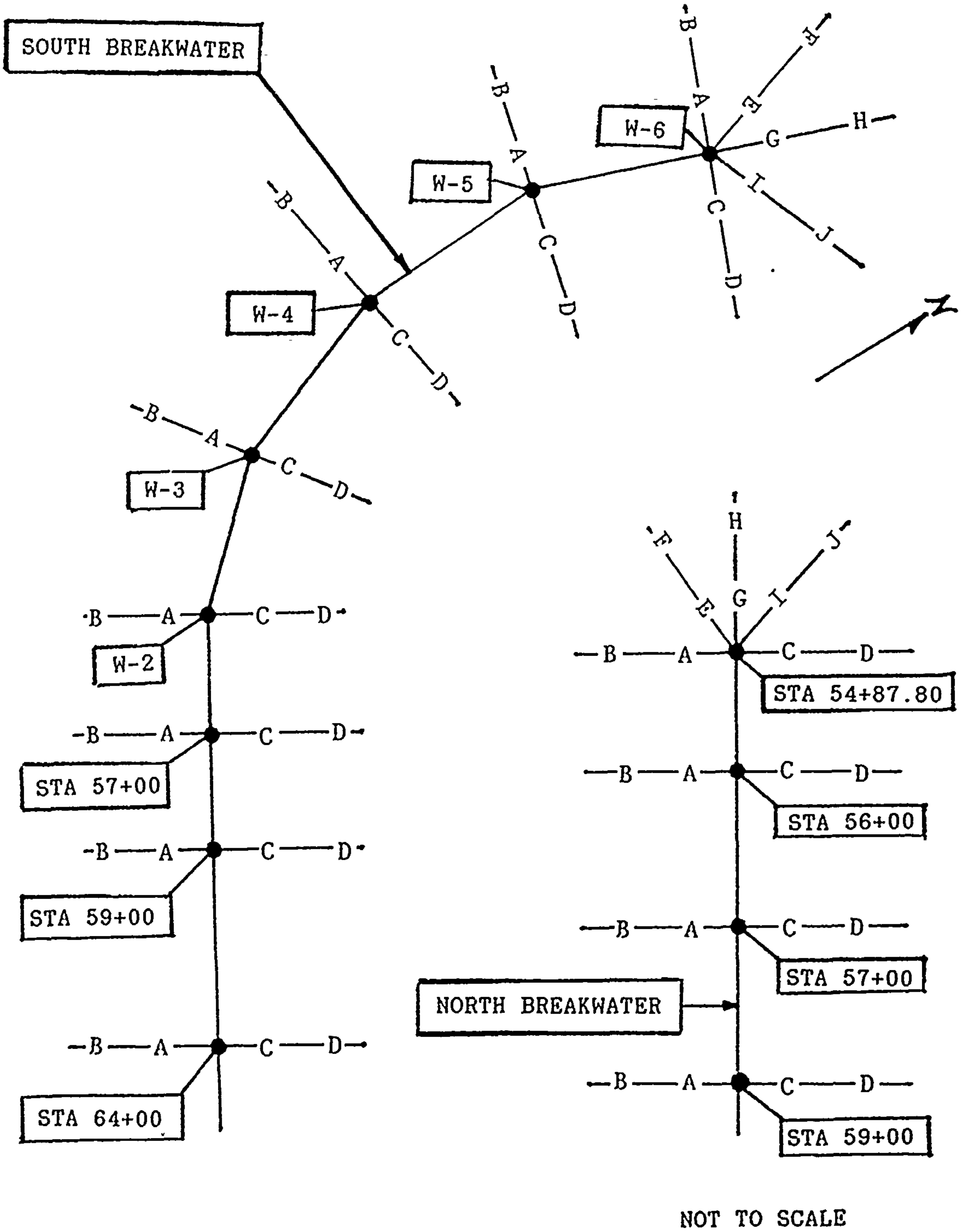


Figure 27. Sketch of target and surveyed stone locations

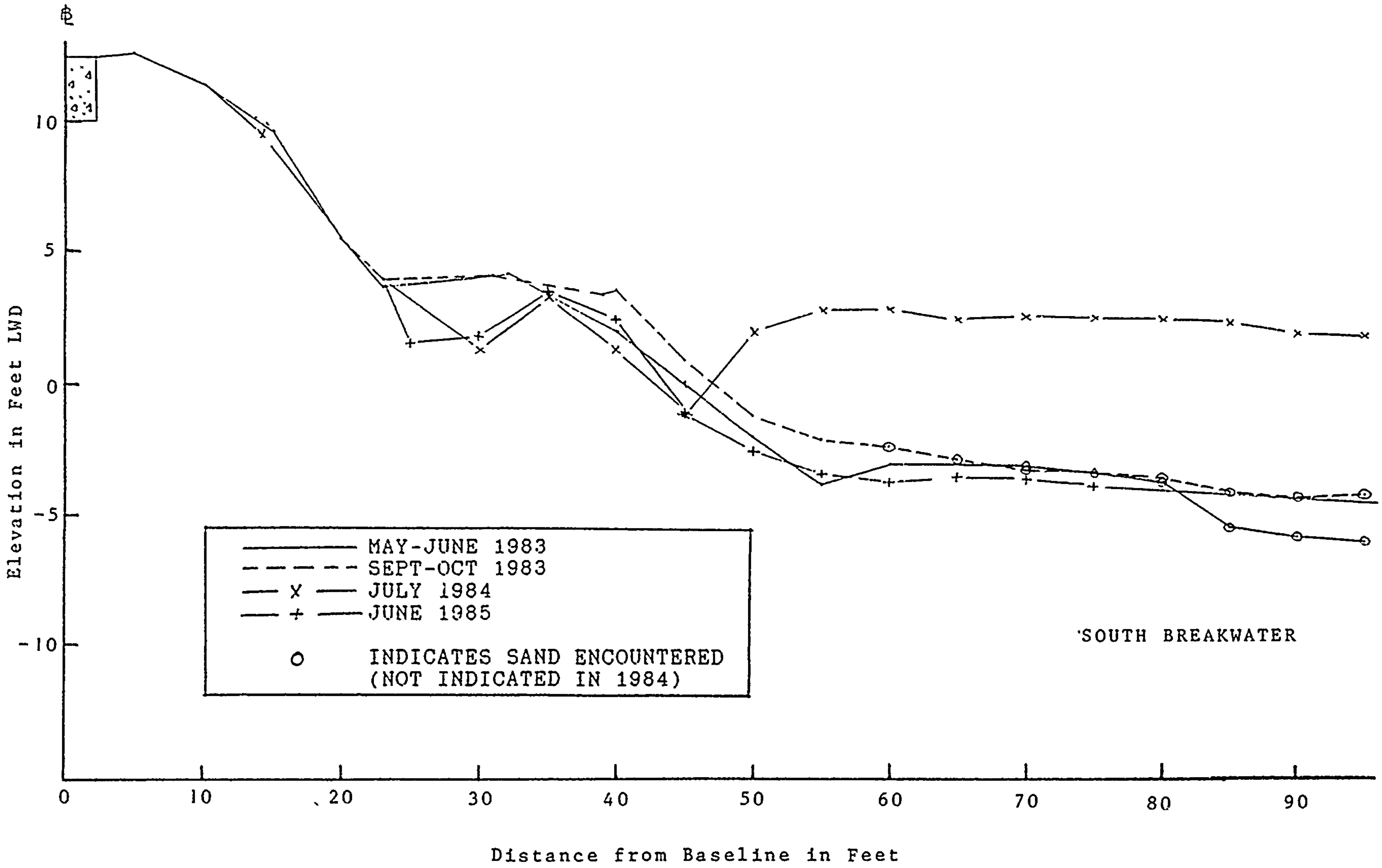


Figure 28. Structure cross section for Sta 57+00 (lakeside)

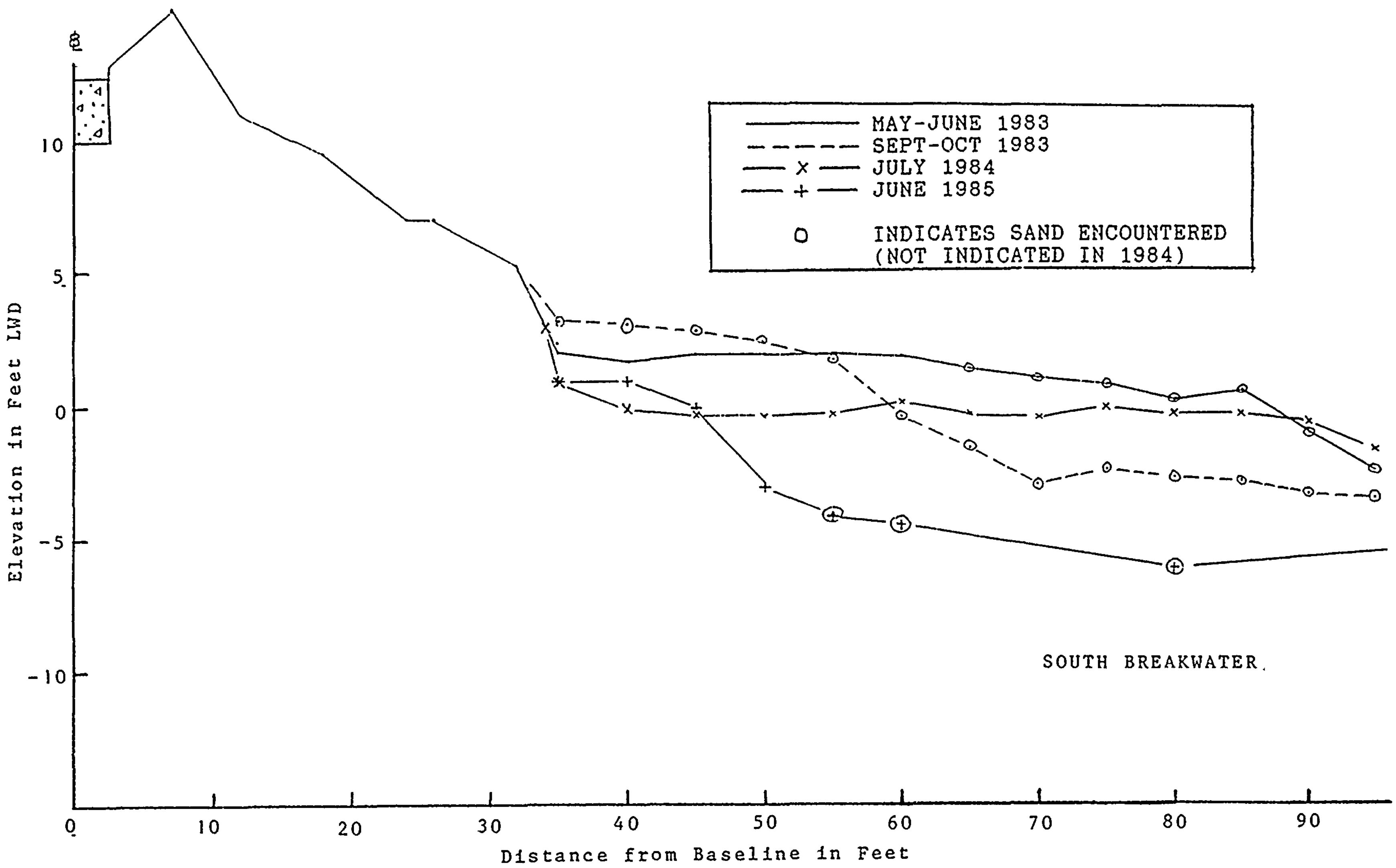


Figure 29. Structure cross section for Sta 57+00 (channelside)

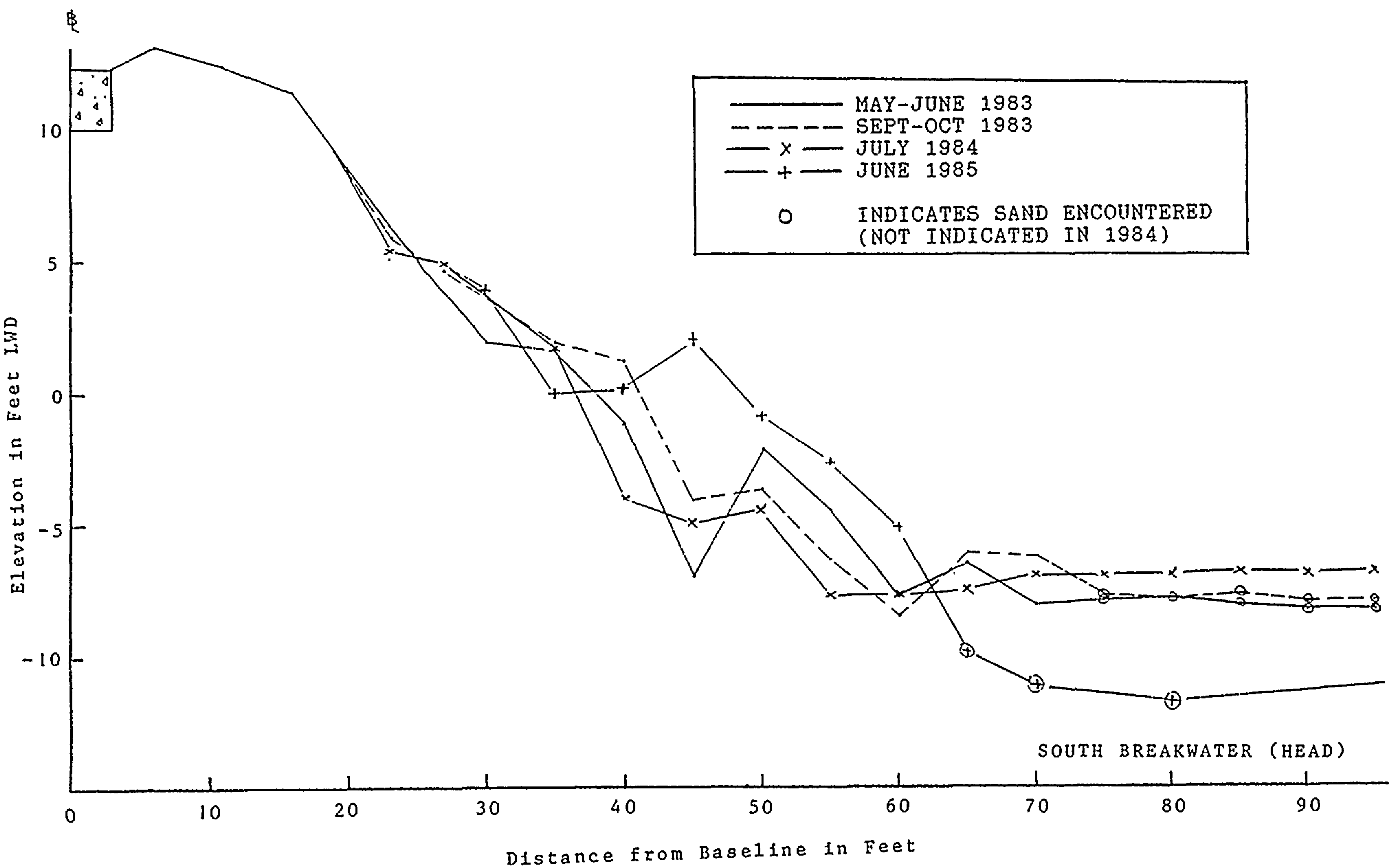


Figure 30. Structure cross section for W-6 (NW)

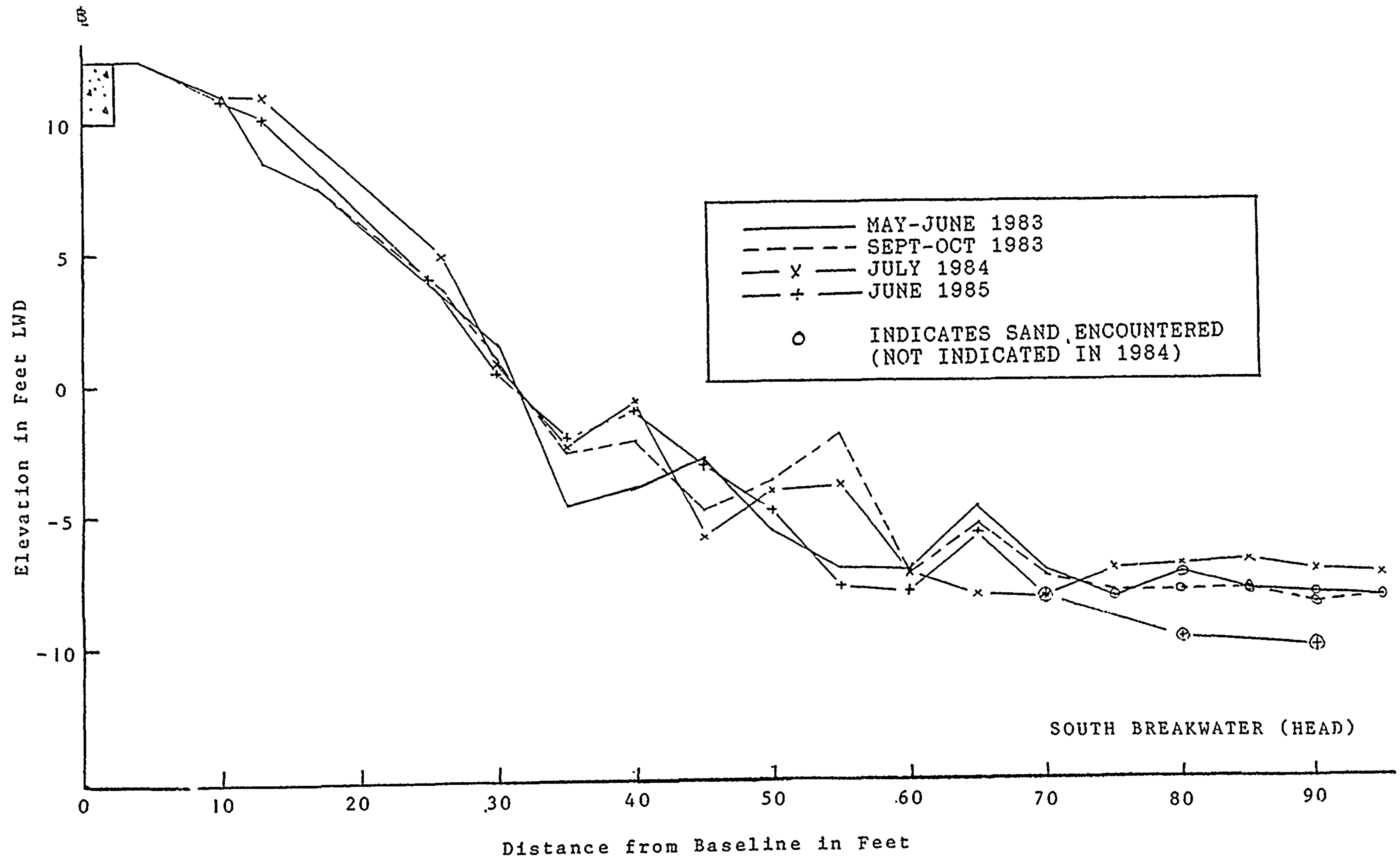


Figure 31. Structure cross section for W-6 (W)

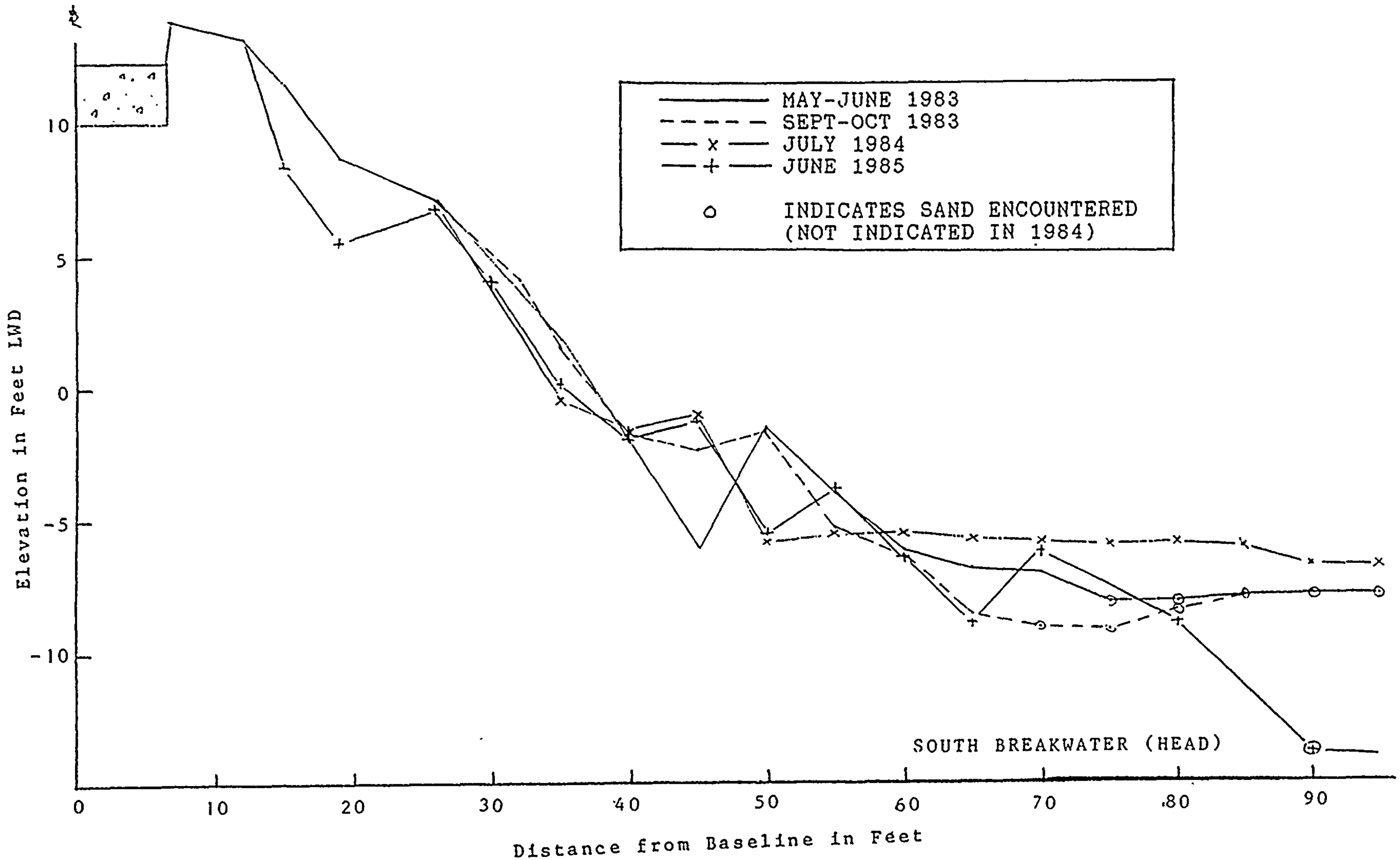


Figure 32. Structure cross section for W-6 (N)

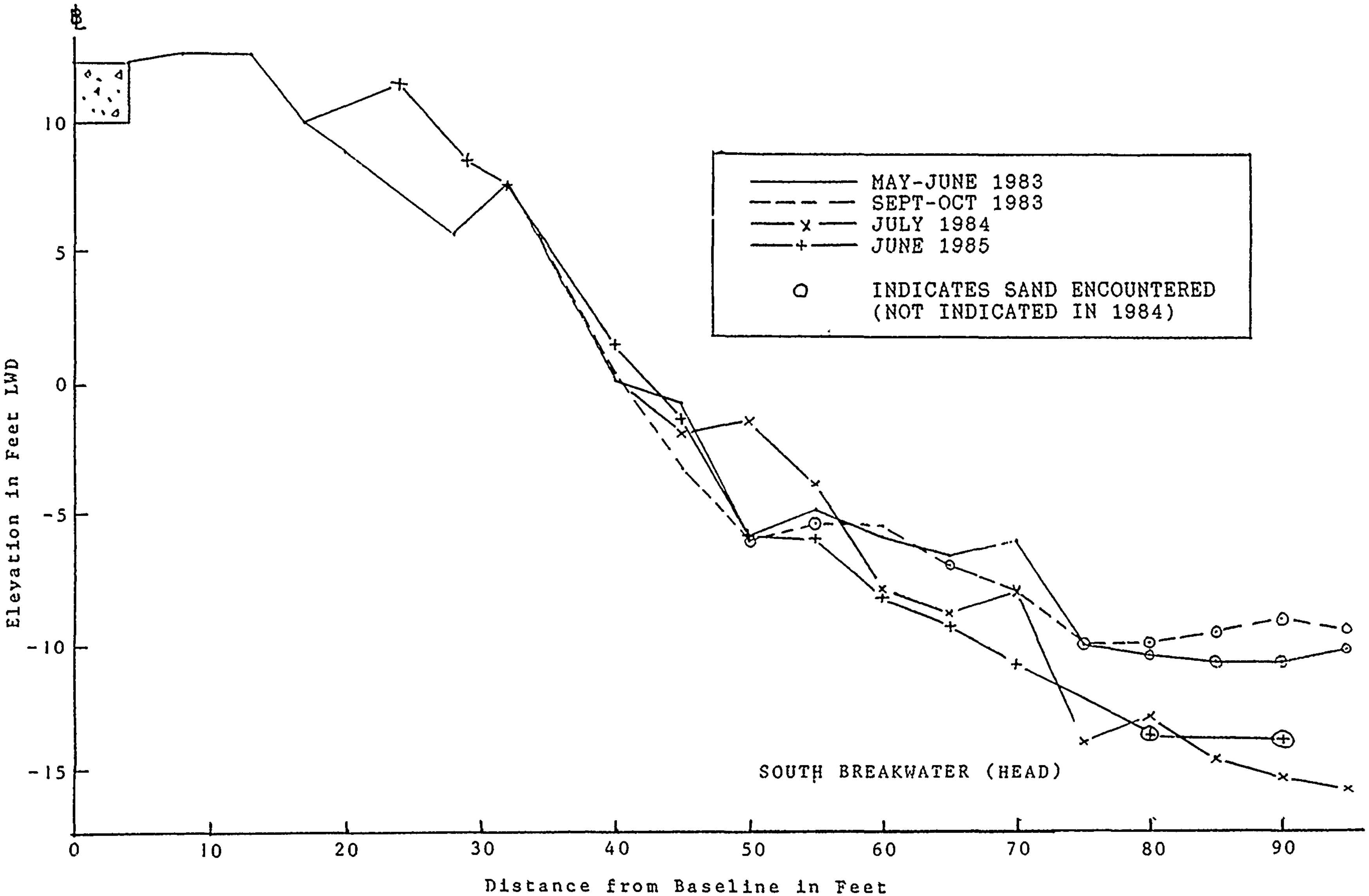


Figure 33. Structure cross section for W-6 (NE)

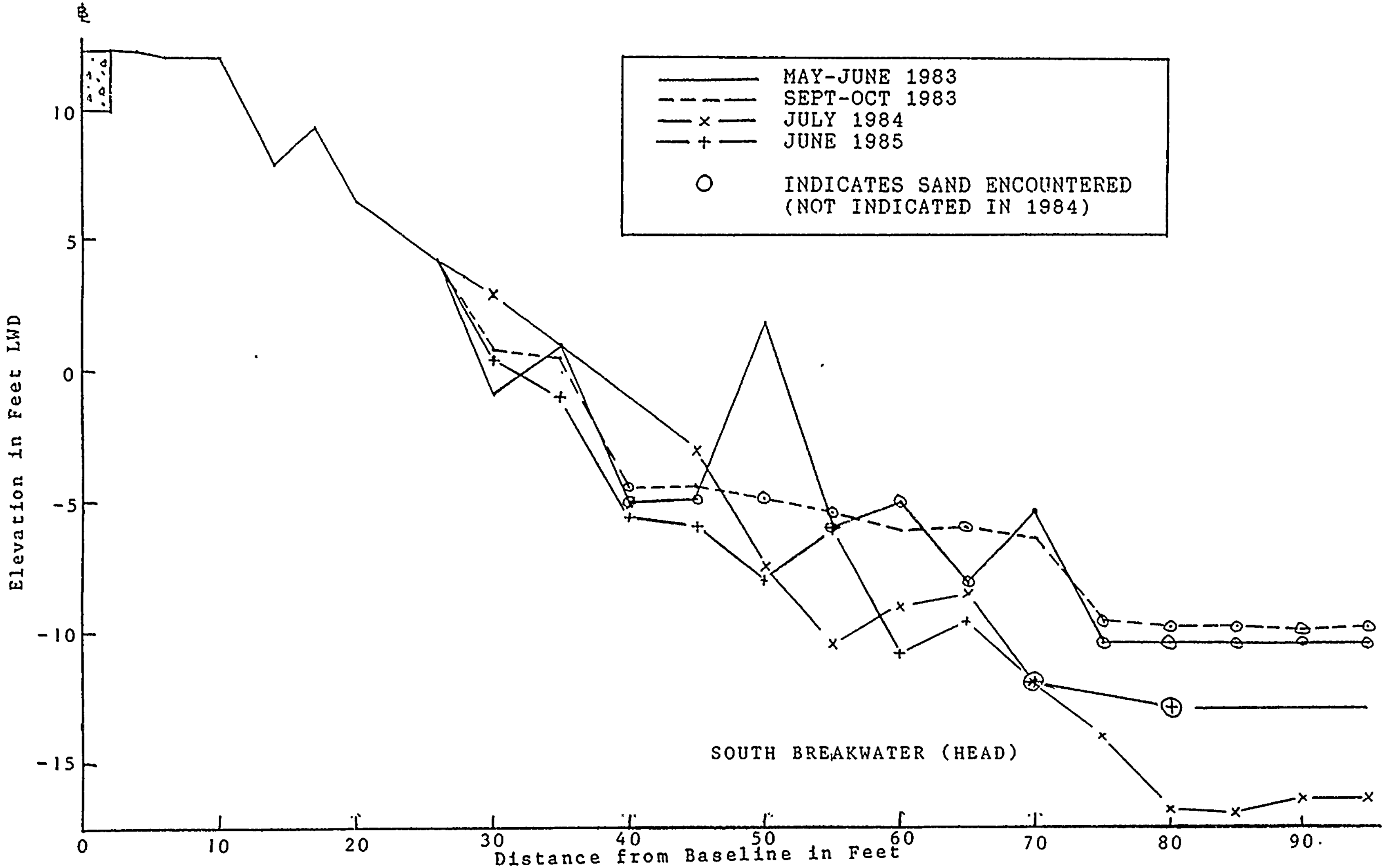


Figure 34. Structure cross section for W-6 (E)

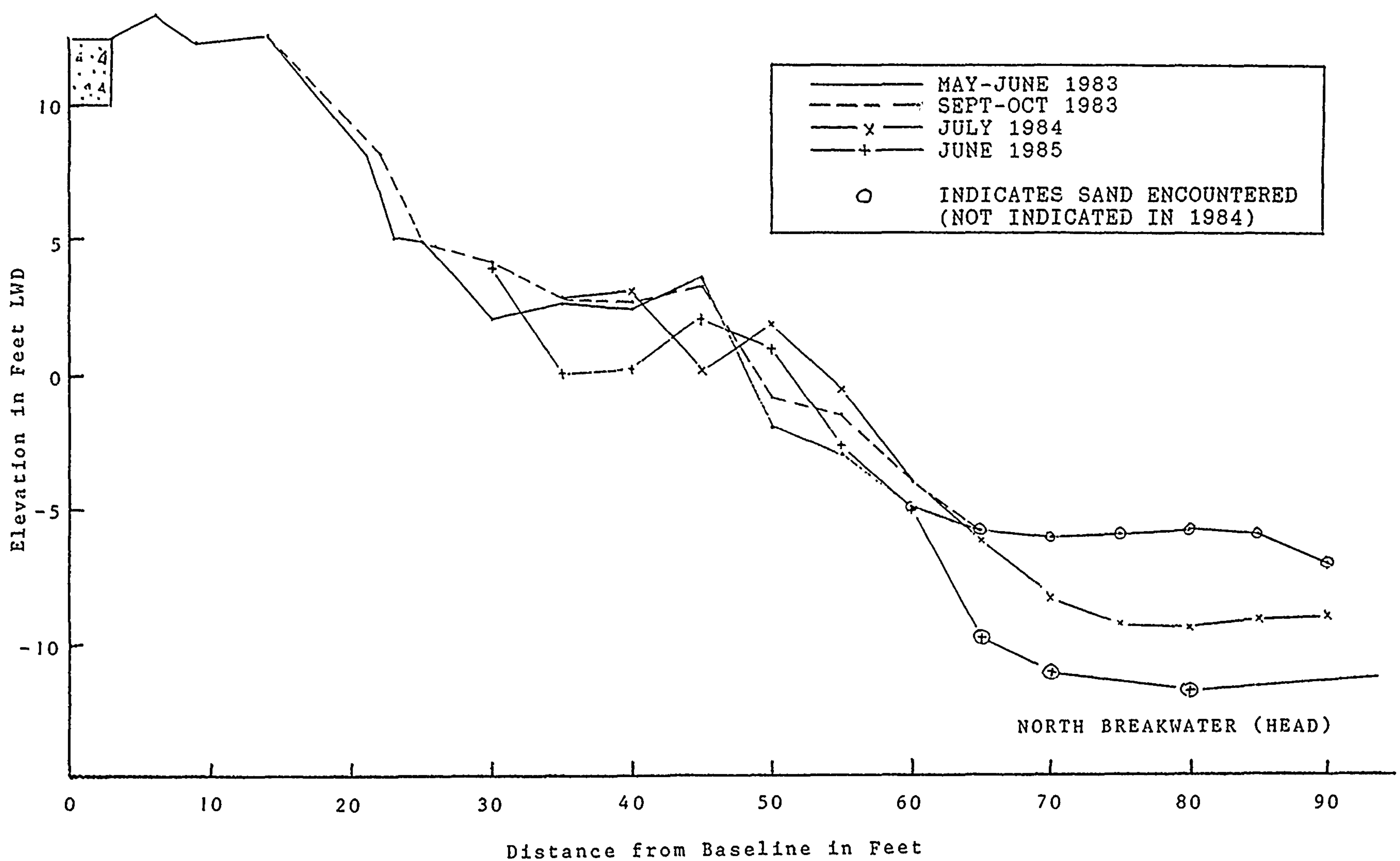


Figure 35. Structure cross section for Sta 54+87.8 (W)

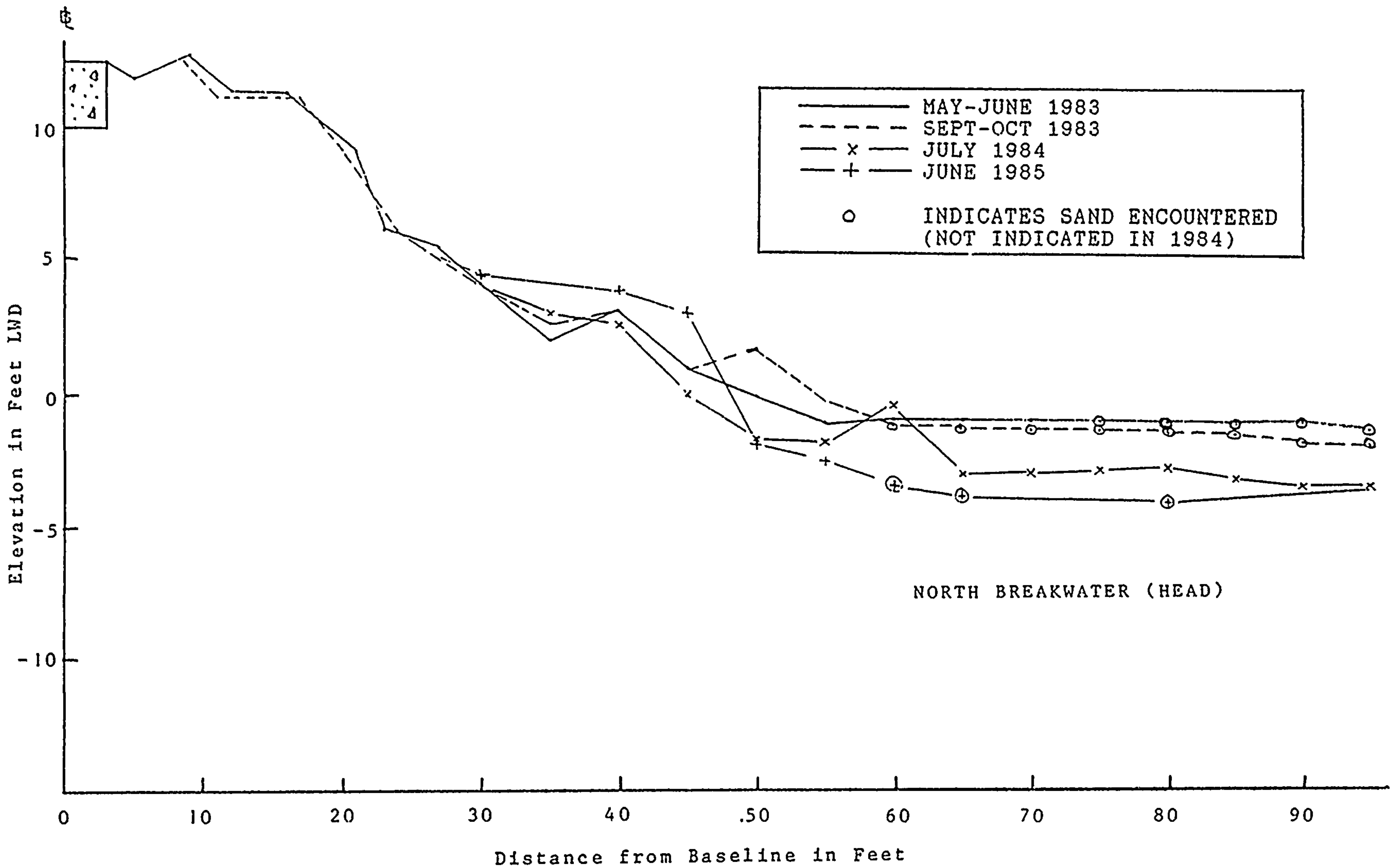


Figure 36. Structure cross section for Sta 54+87.8 (N)

Figure 37. South breakwater, lakeside, 17 November 1988

Figure 38. South breakwater, channelside, 17 November 1988

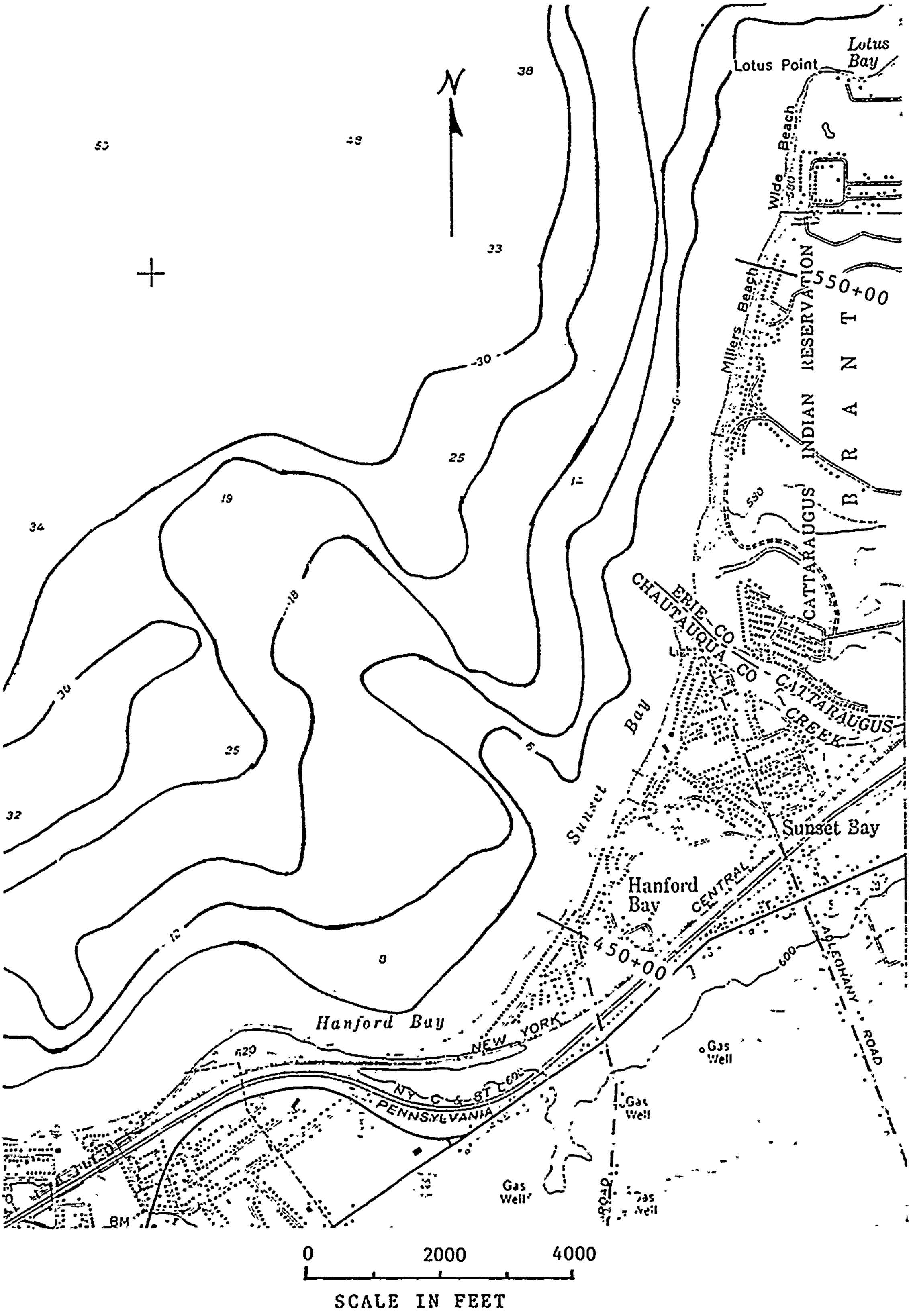


Figure 39. Offshore bathymetry in 1960

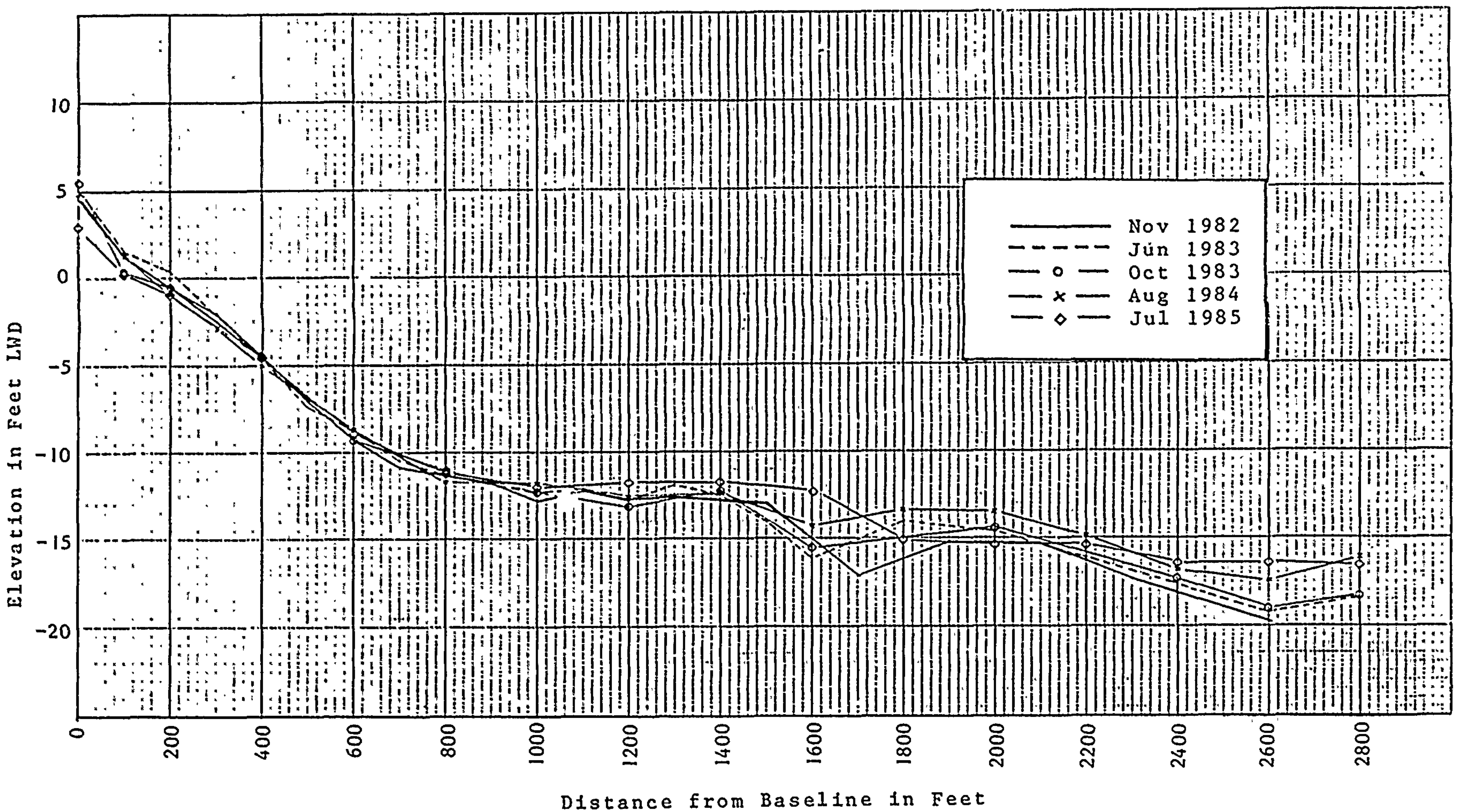


Figure 40. Profile comparisons for Sta 450+00

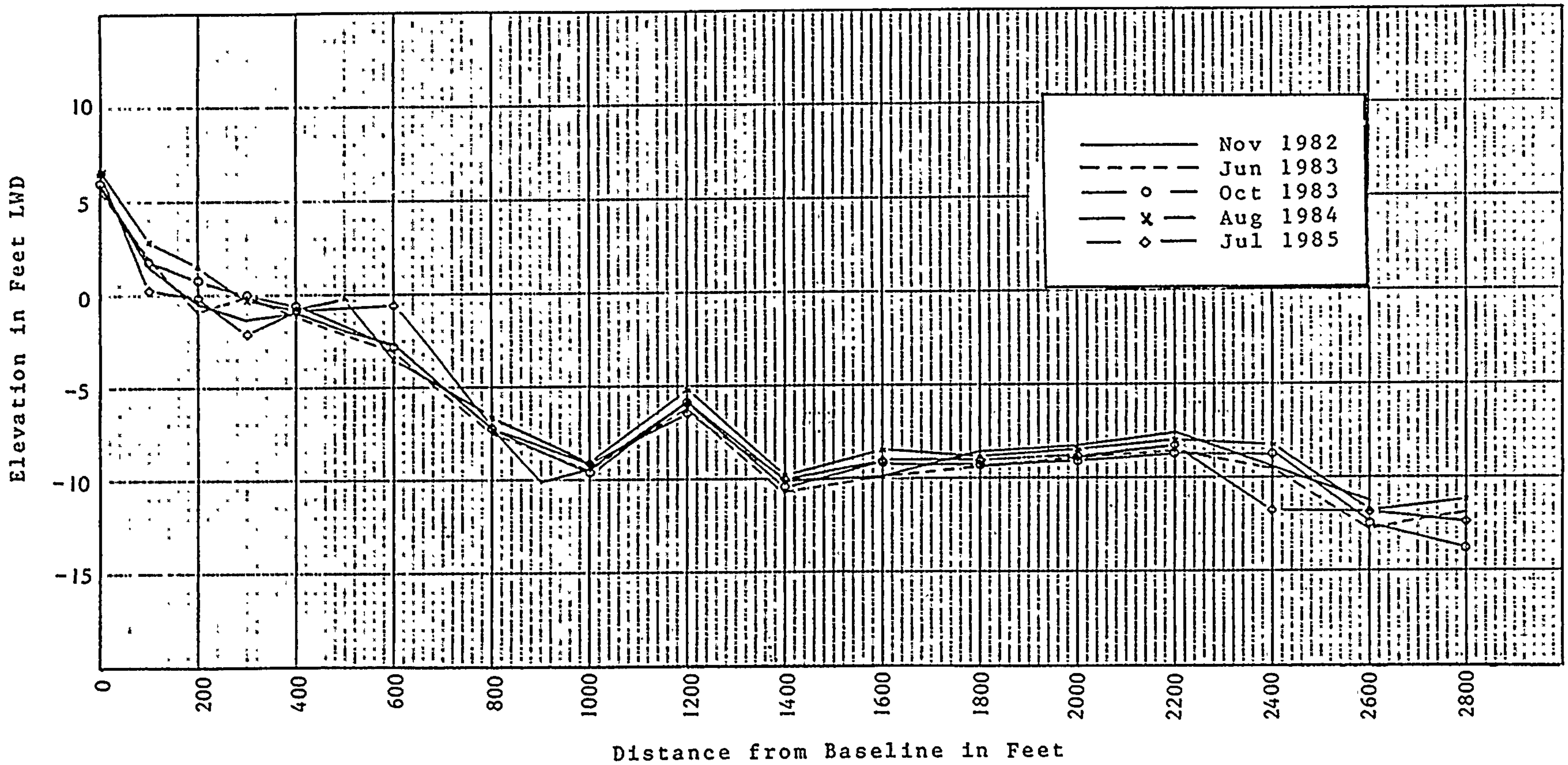


Figure 41. Profile comparisons for Sta 465+00

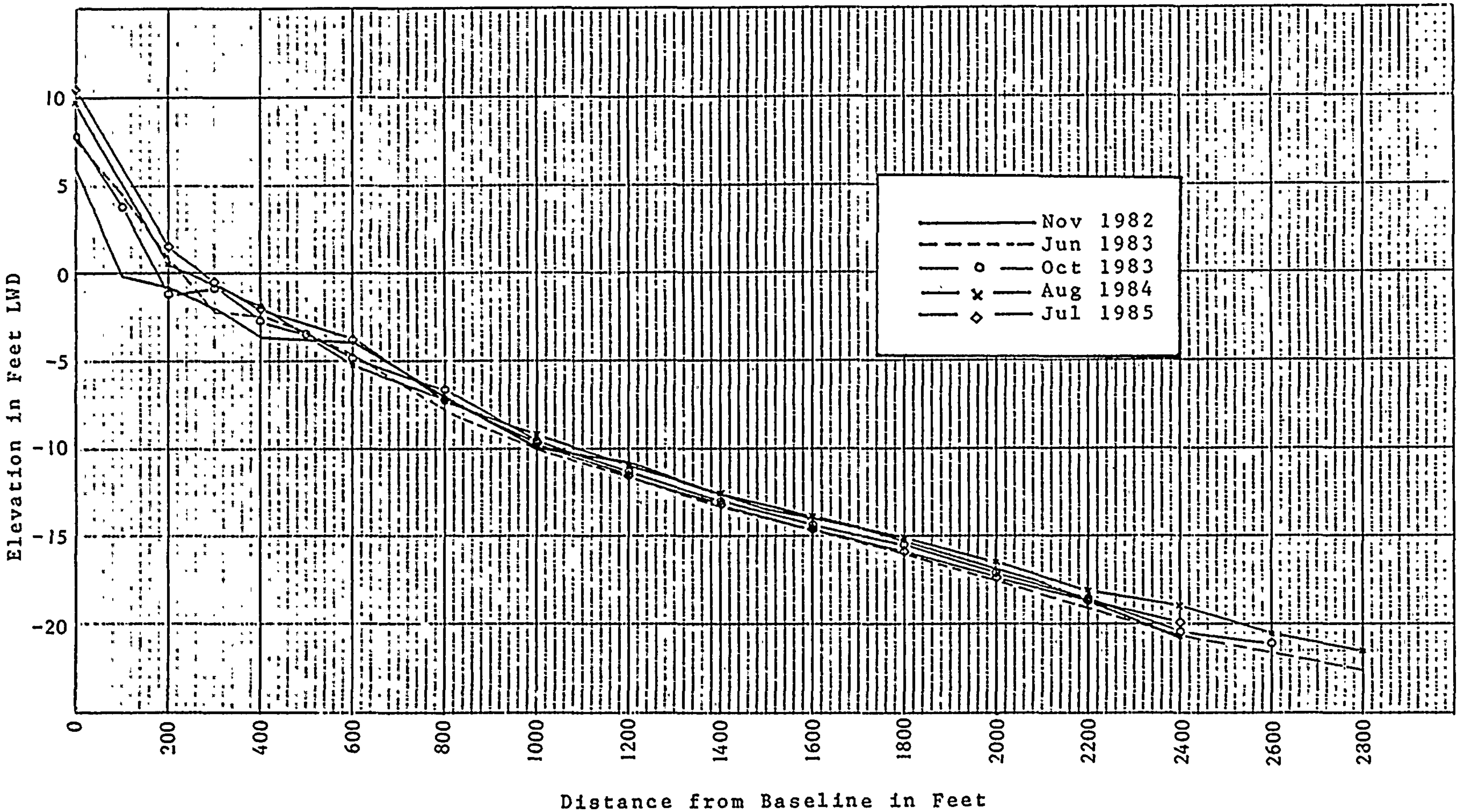


Figure 42. Profile comparisons for Sta 485+00

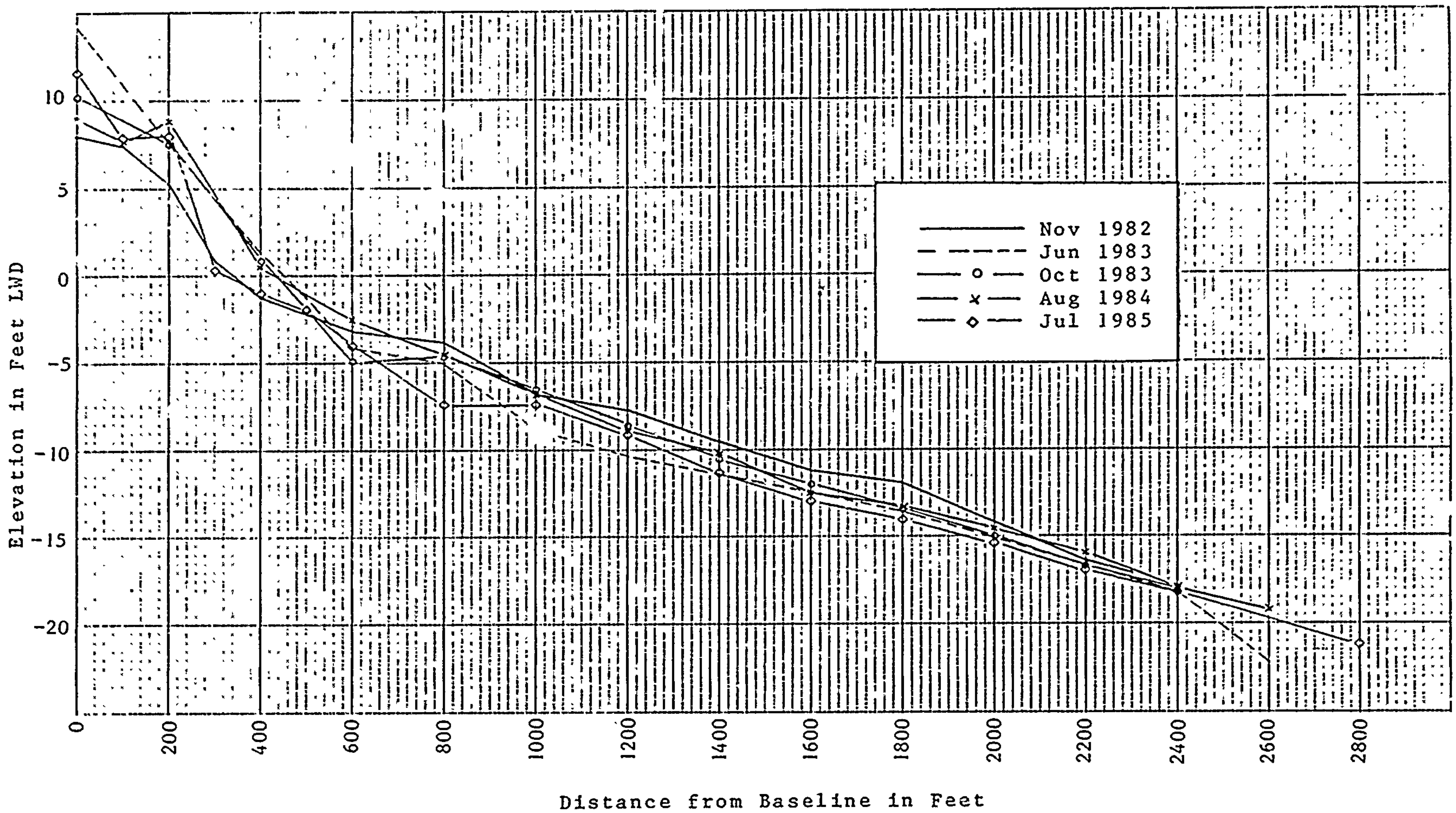


Figure 43. Profile comparisons for Sta 490+00

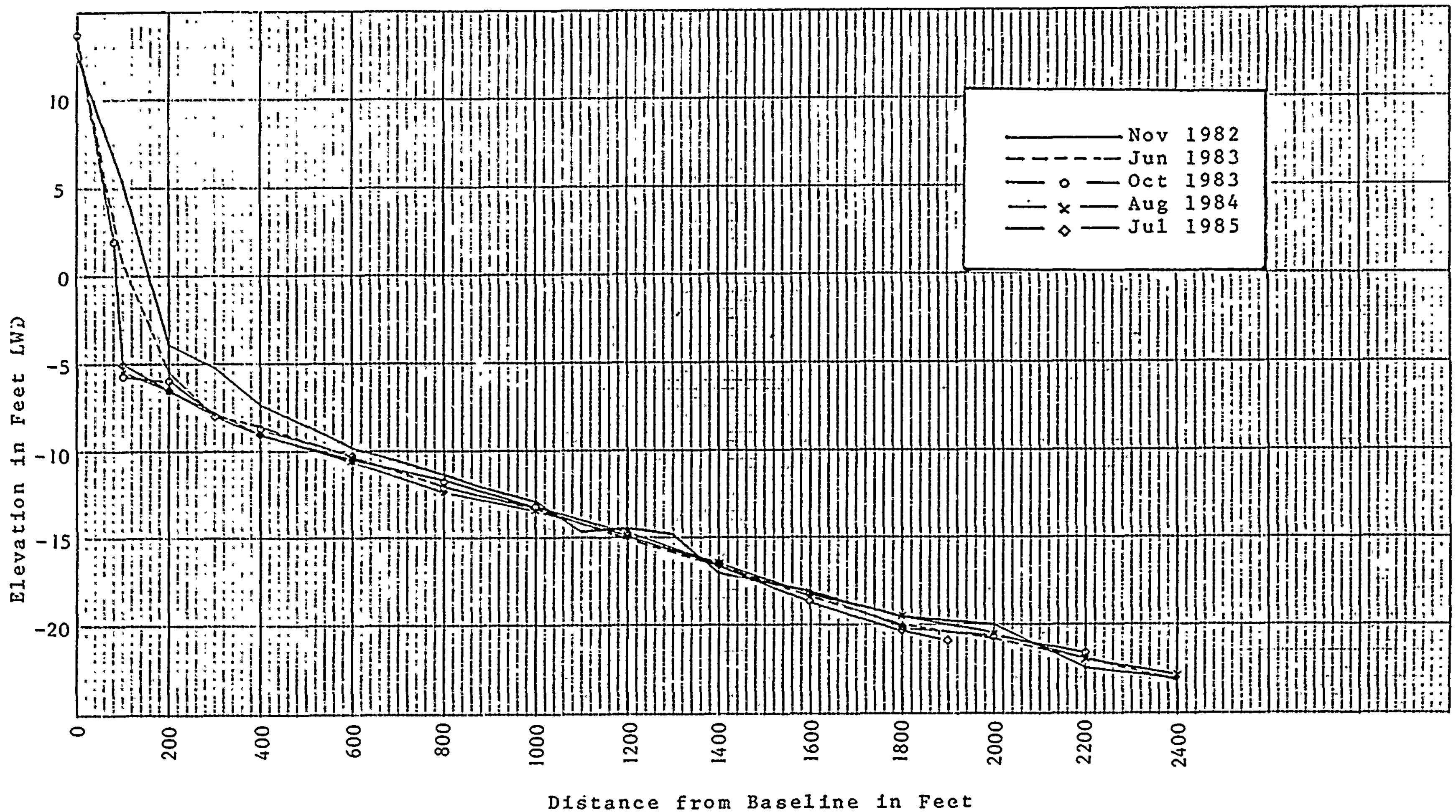


Figure 44. Profile comparisons for Sta 495+00

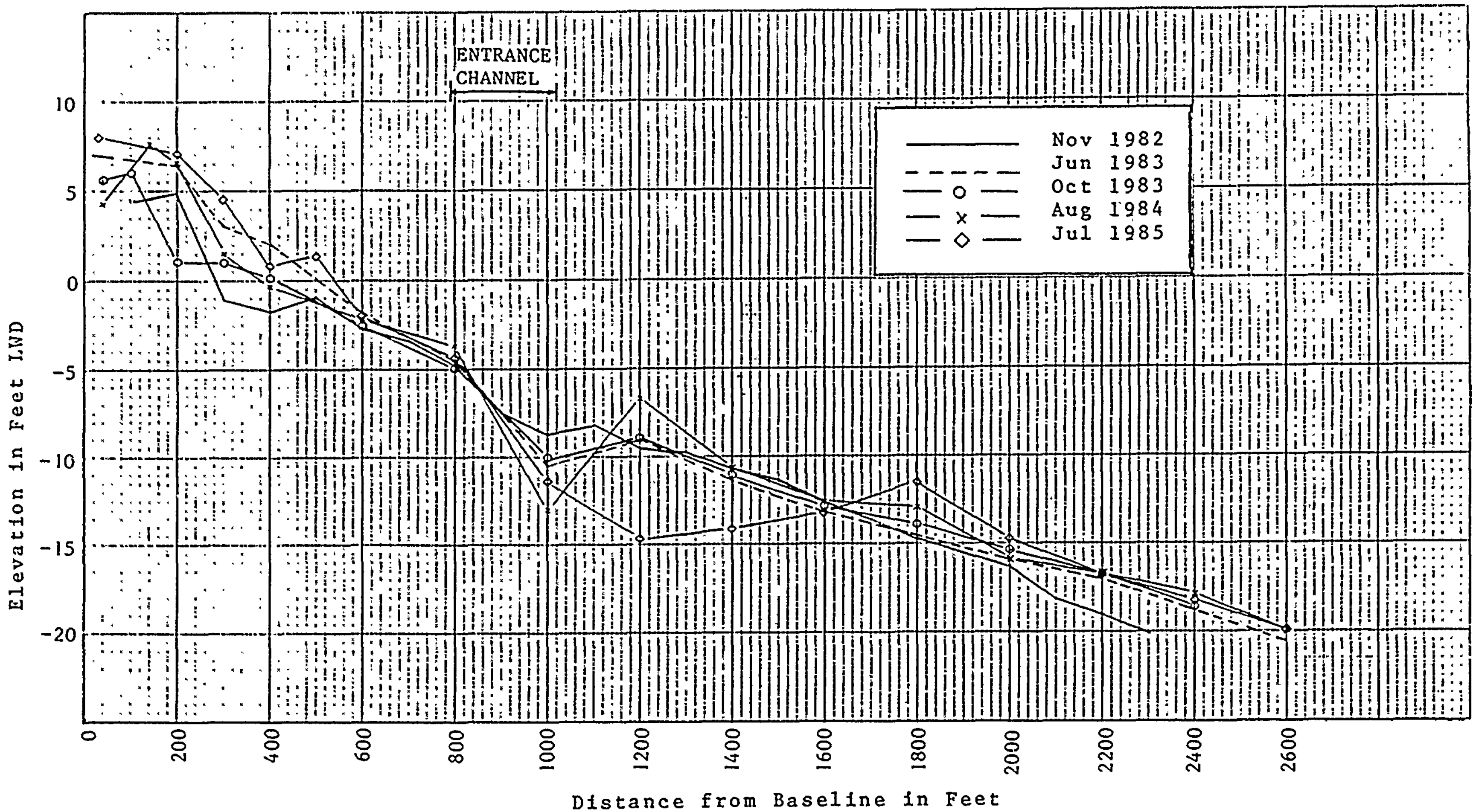


Figure 45. Profile comparisons for Sta 501+00

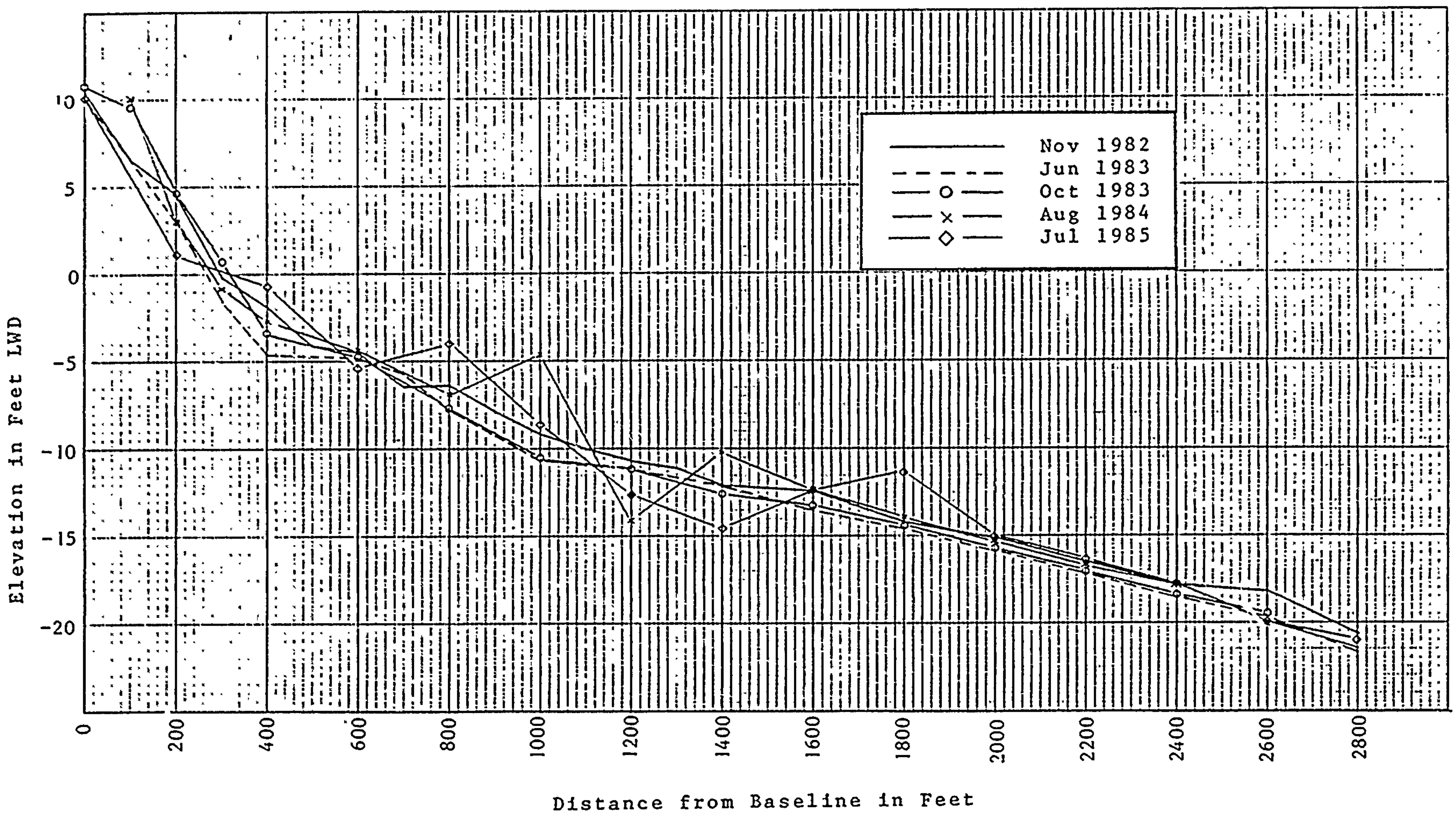


Figure 46. Profile comparisons for Sta 505+00

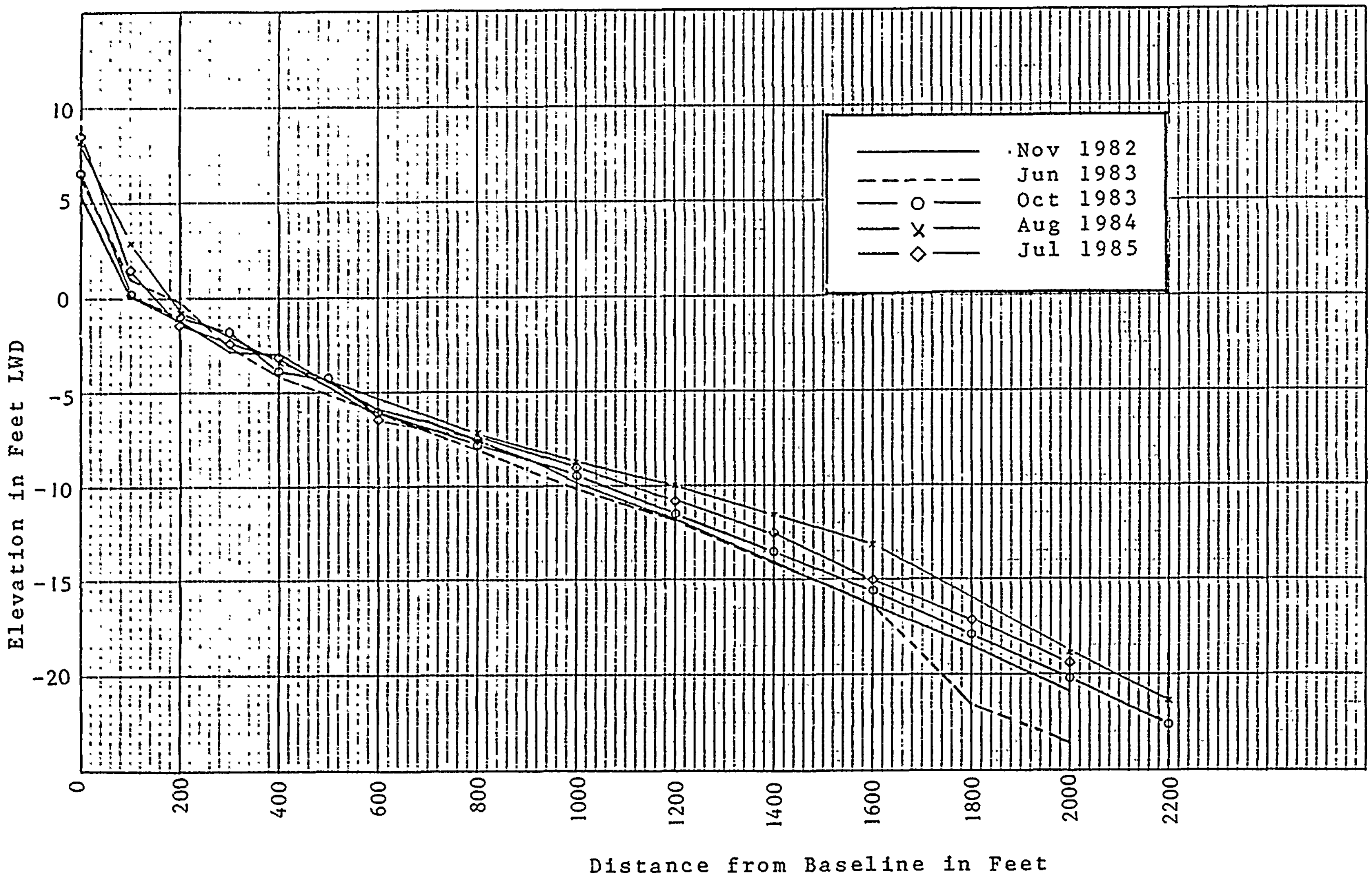


Figure 47. Profile comparisons for Sta 525+00

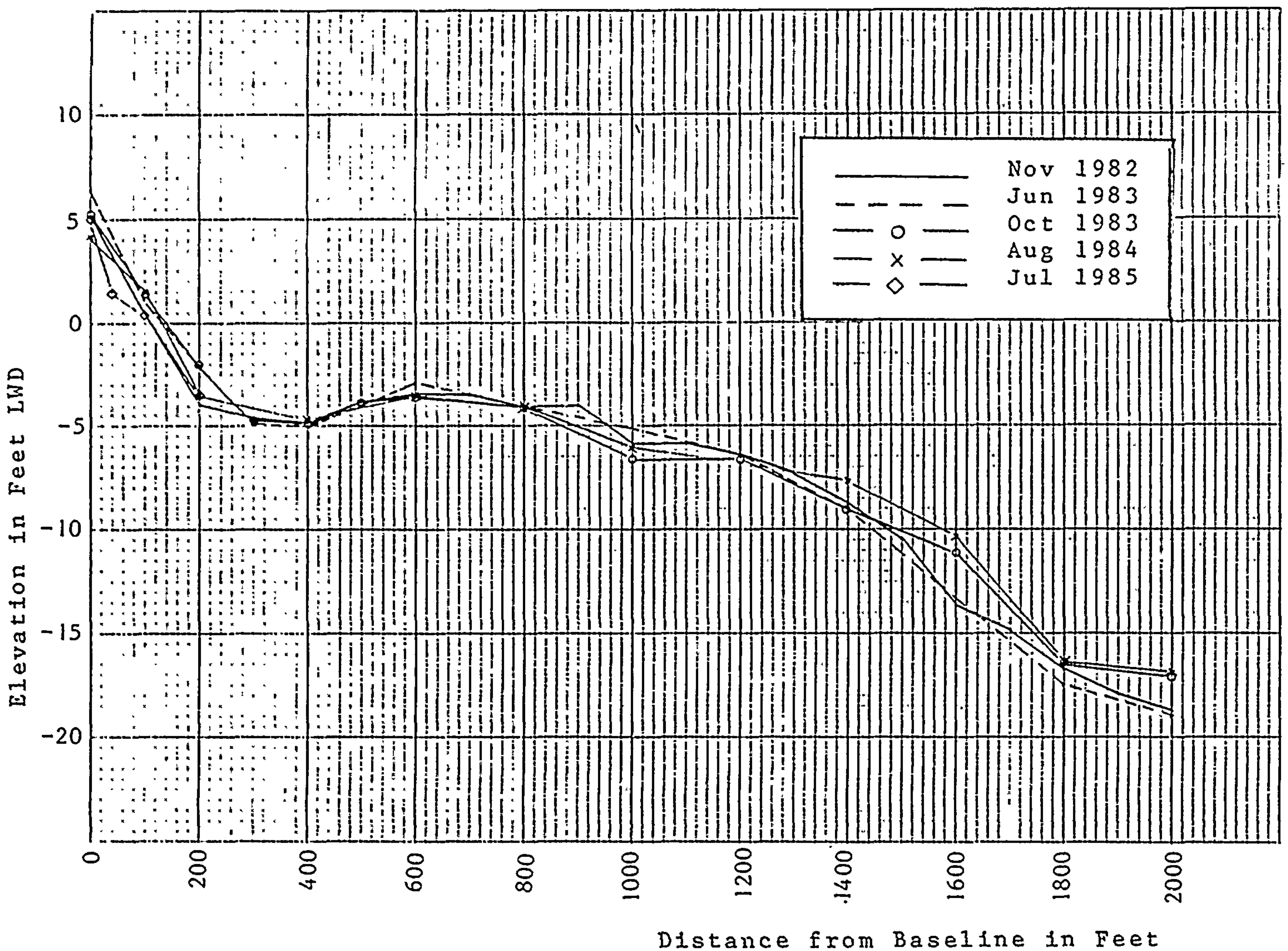


Figure 48. Profile comparisons for Sta 550+00

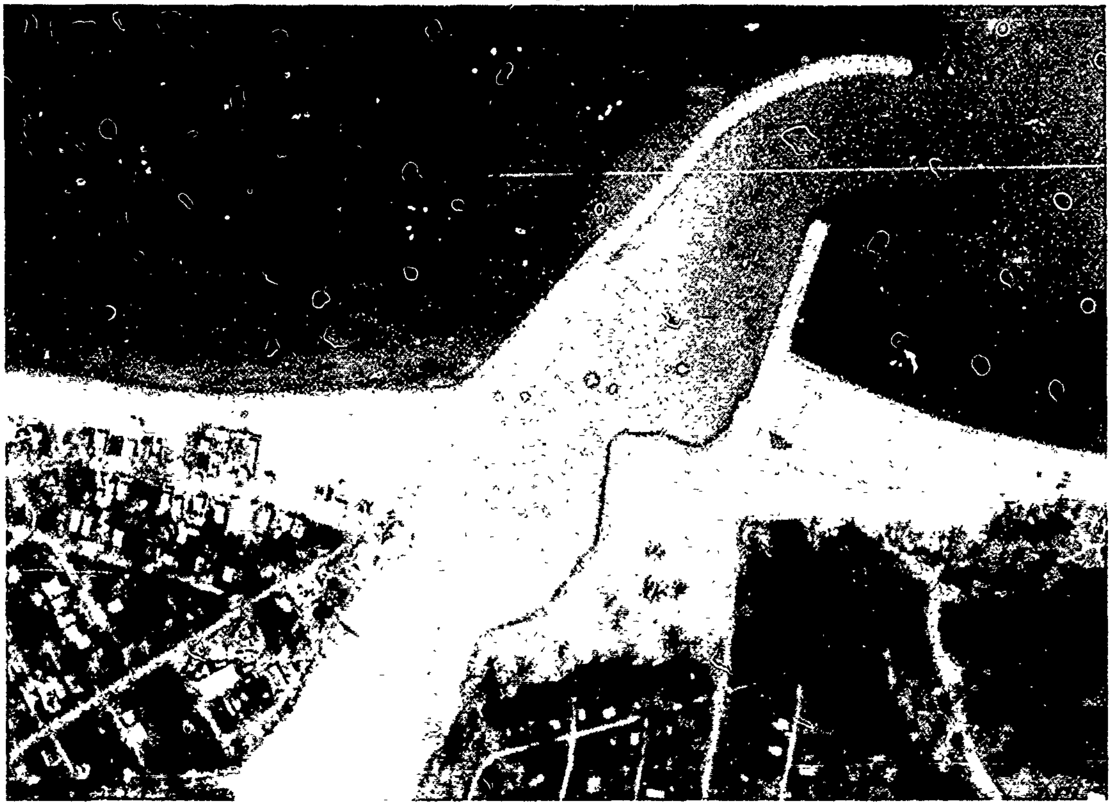


Figure 49. Cattaraugus Creek Harbor,
13 May 1983, +3.6 LWD

Figure 50. Cattaraugus Creek Harbor,
26 October 1985, +3.5 LWD

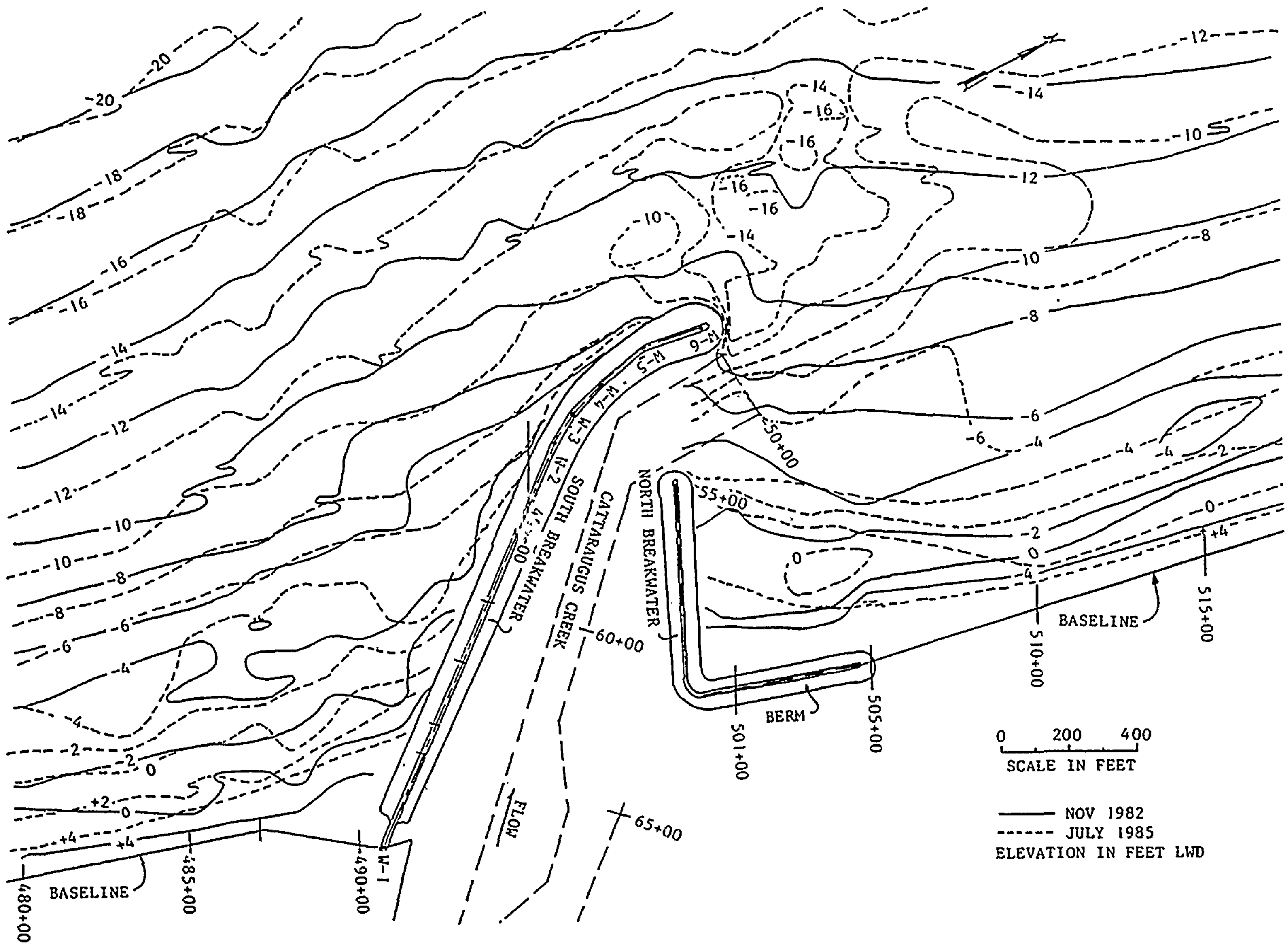


Figure 51. Offshore bathymetry changes in vicinity of breakwaters

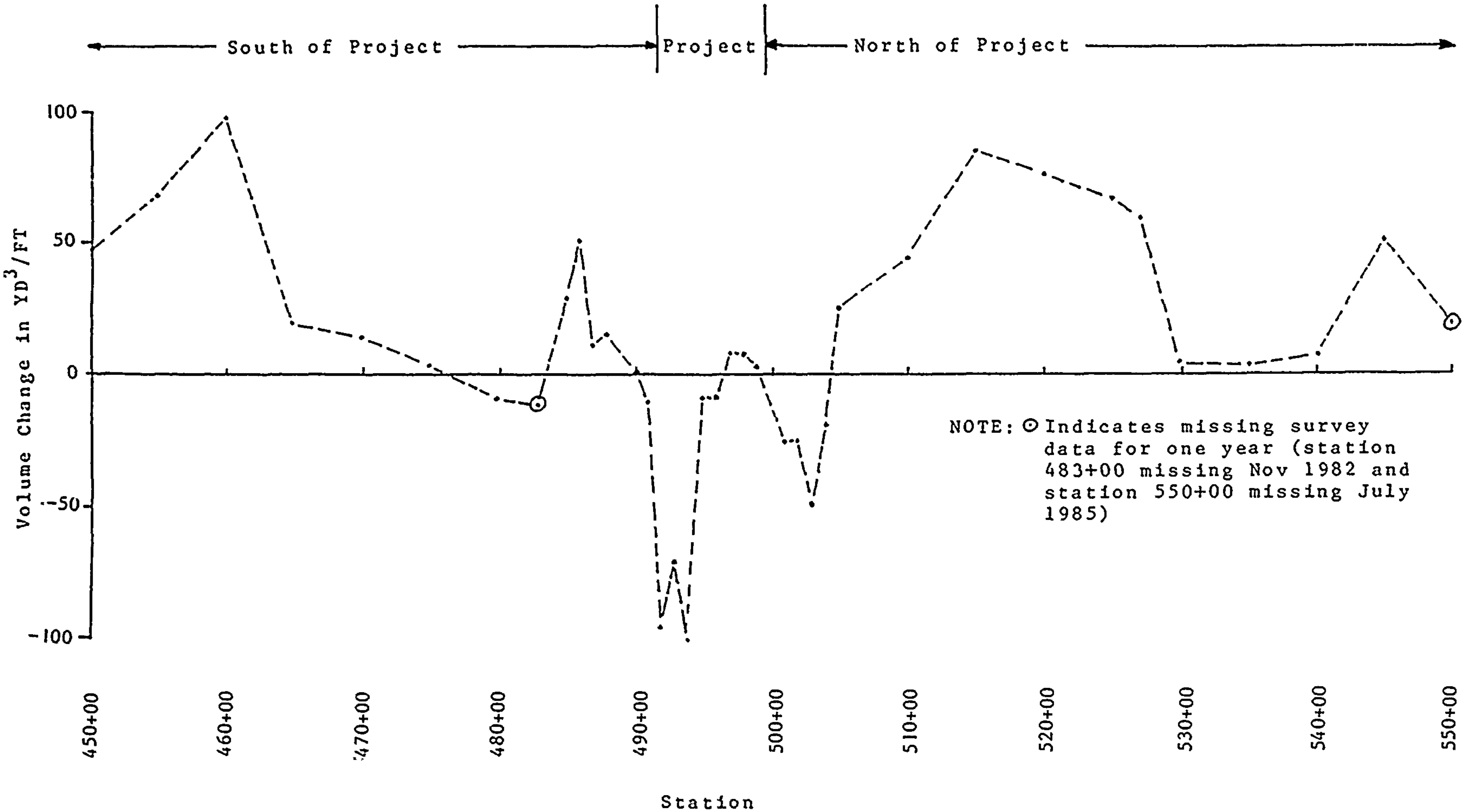


Figure 52. Offshore volume changes (November 1982-July 1985)

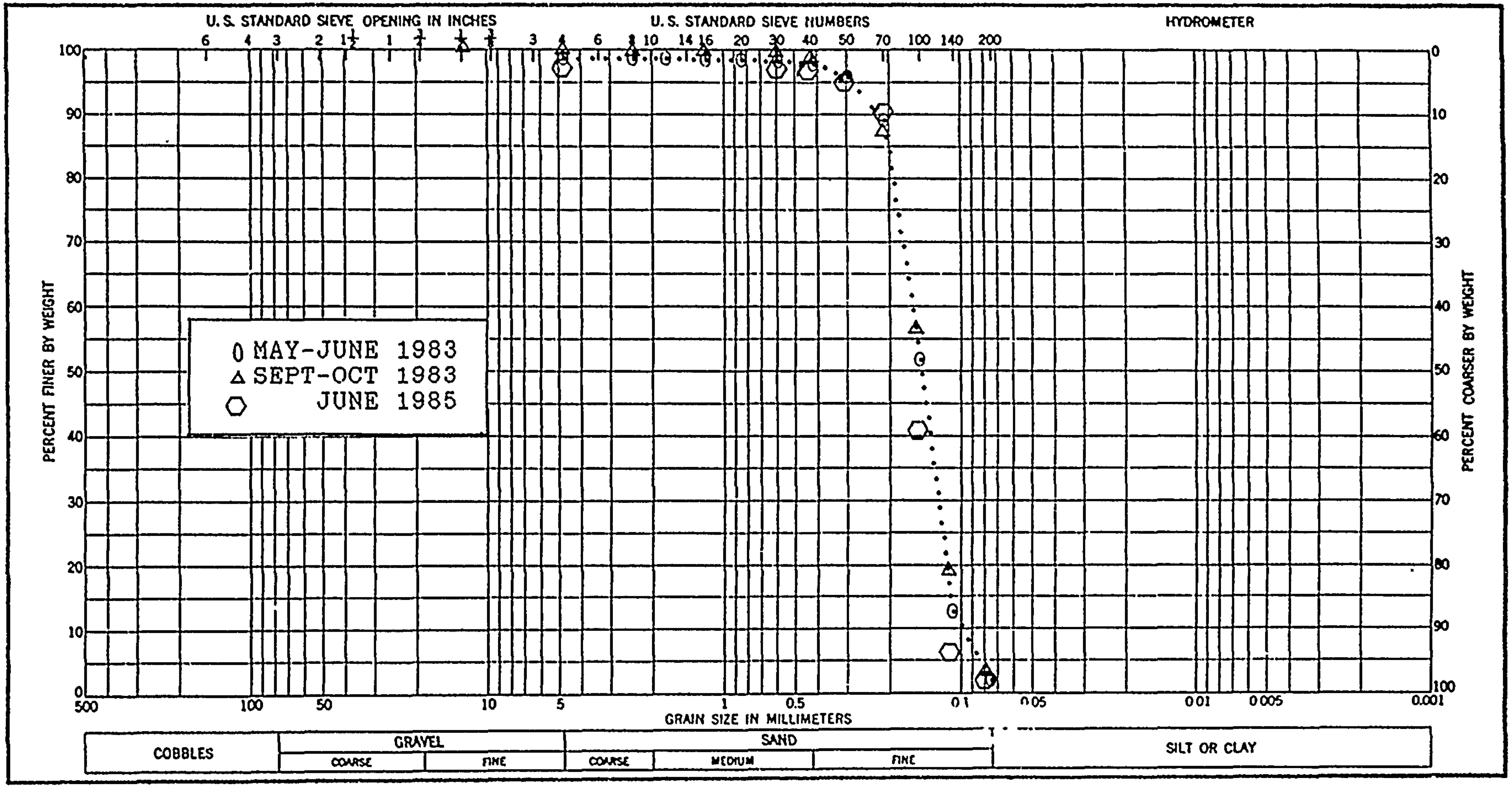


Figure 53. Sediment gradation curves for Sta 480500

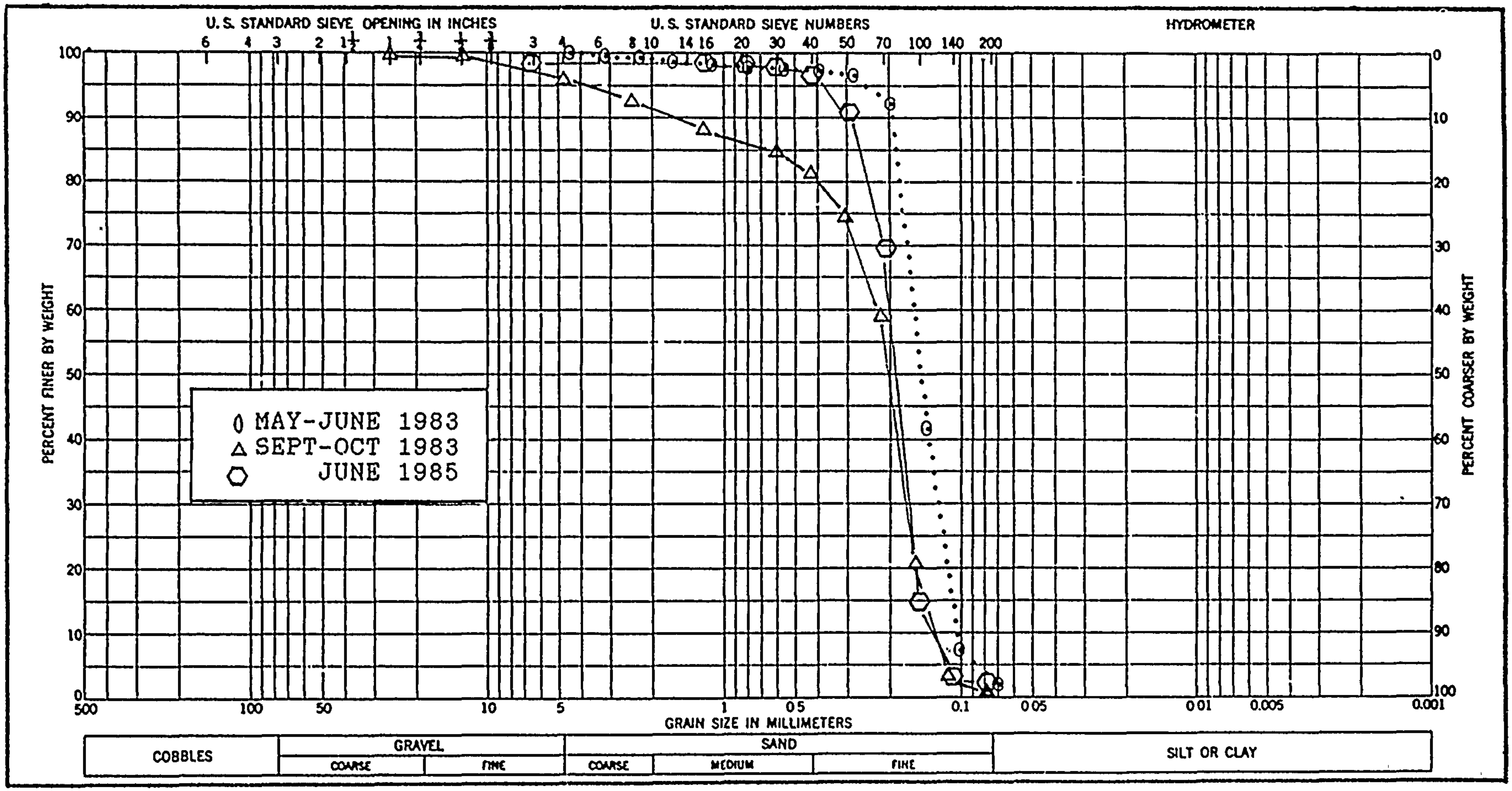


Figure 54. Sediment gradation curves for Sta 491300

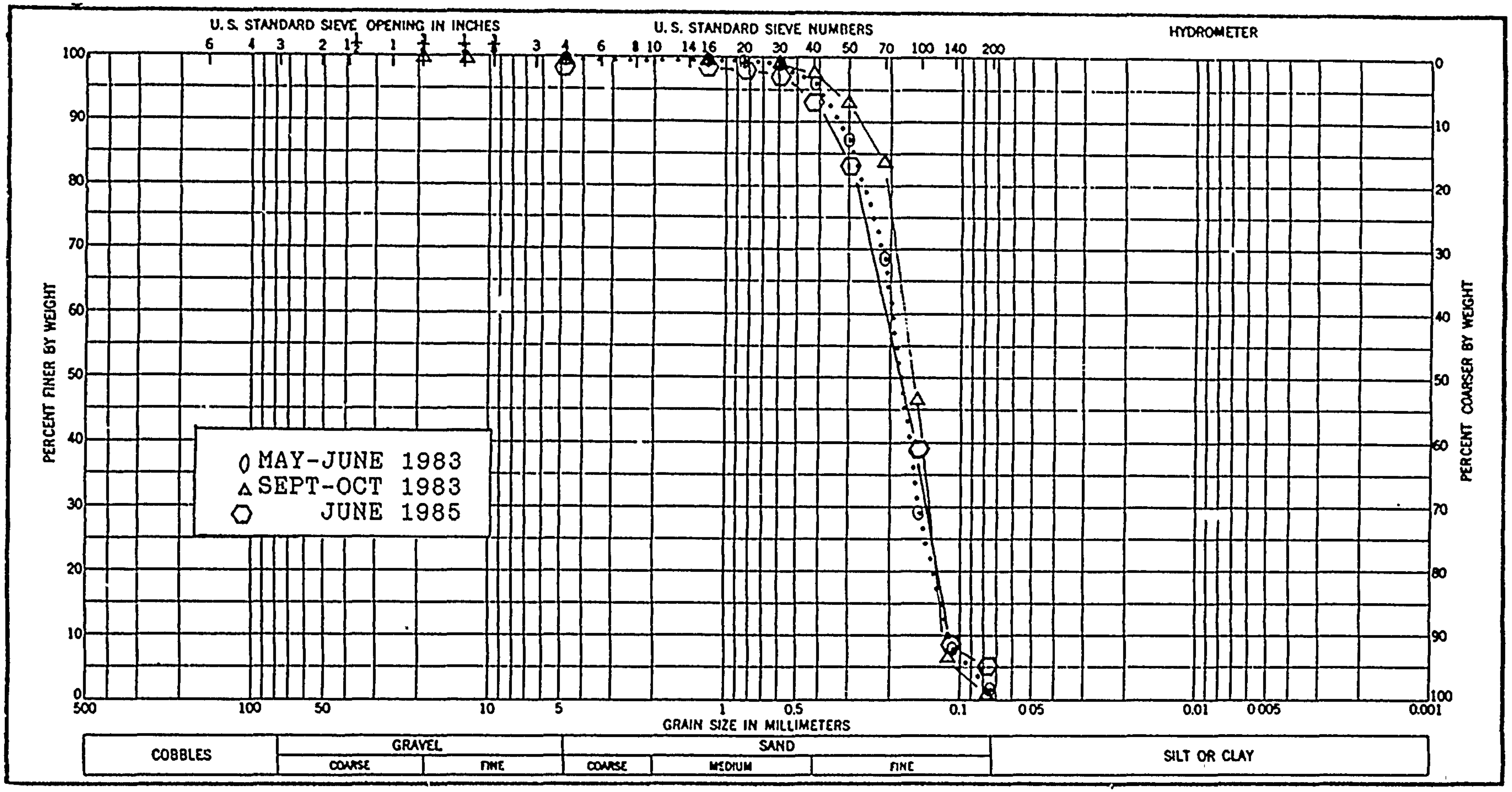


Figure 55. Sediment gradation curves for Sta 491800

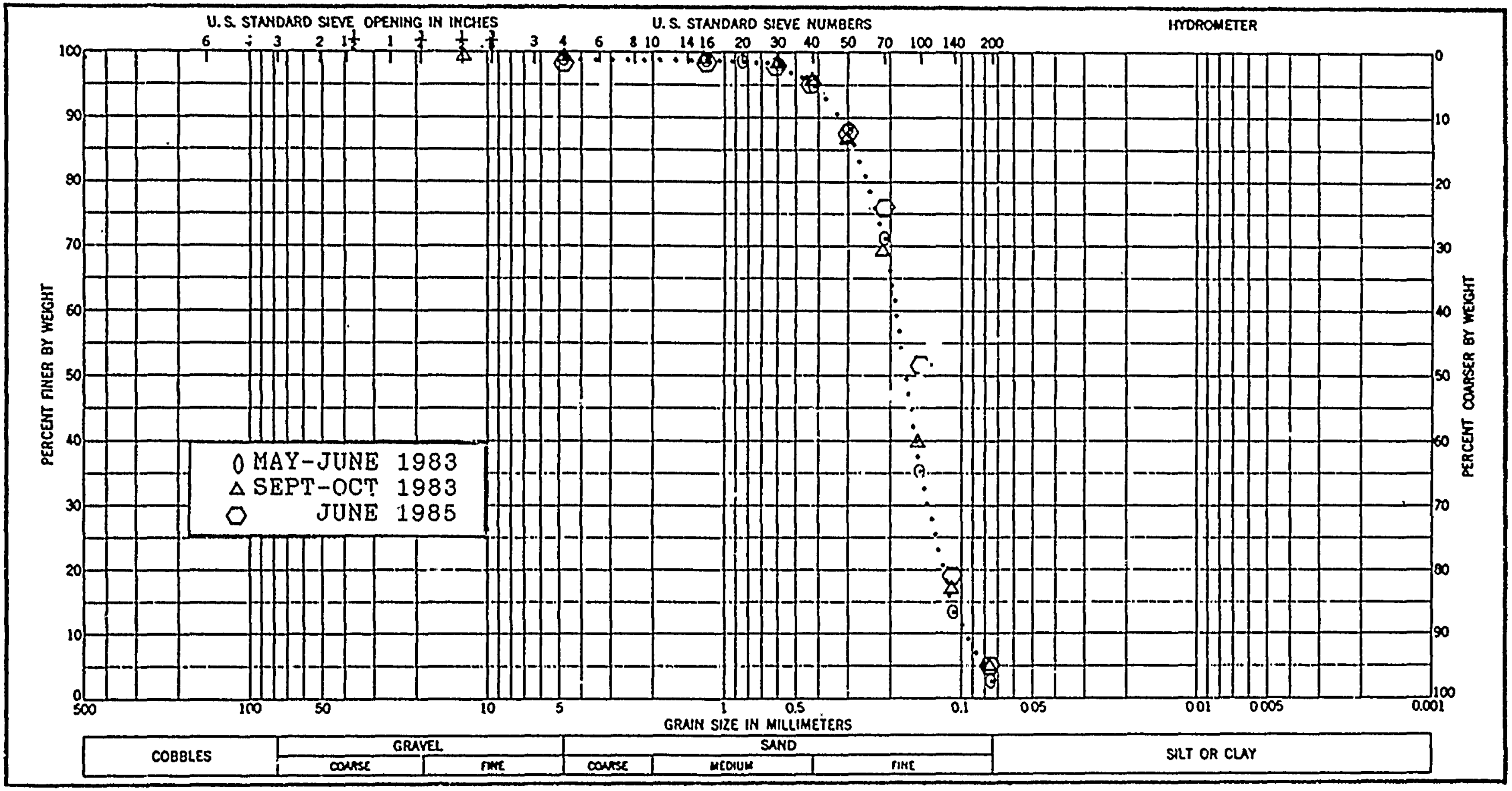


Figure 56. Sediment gradation curves for Sta 4911500

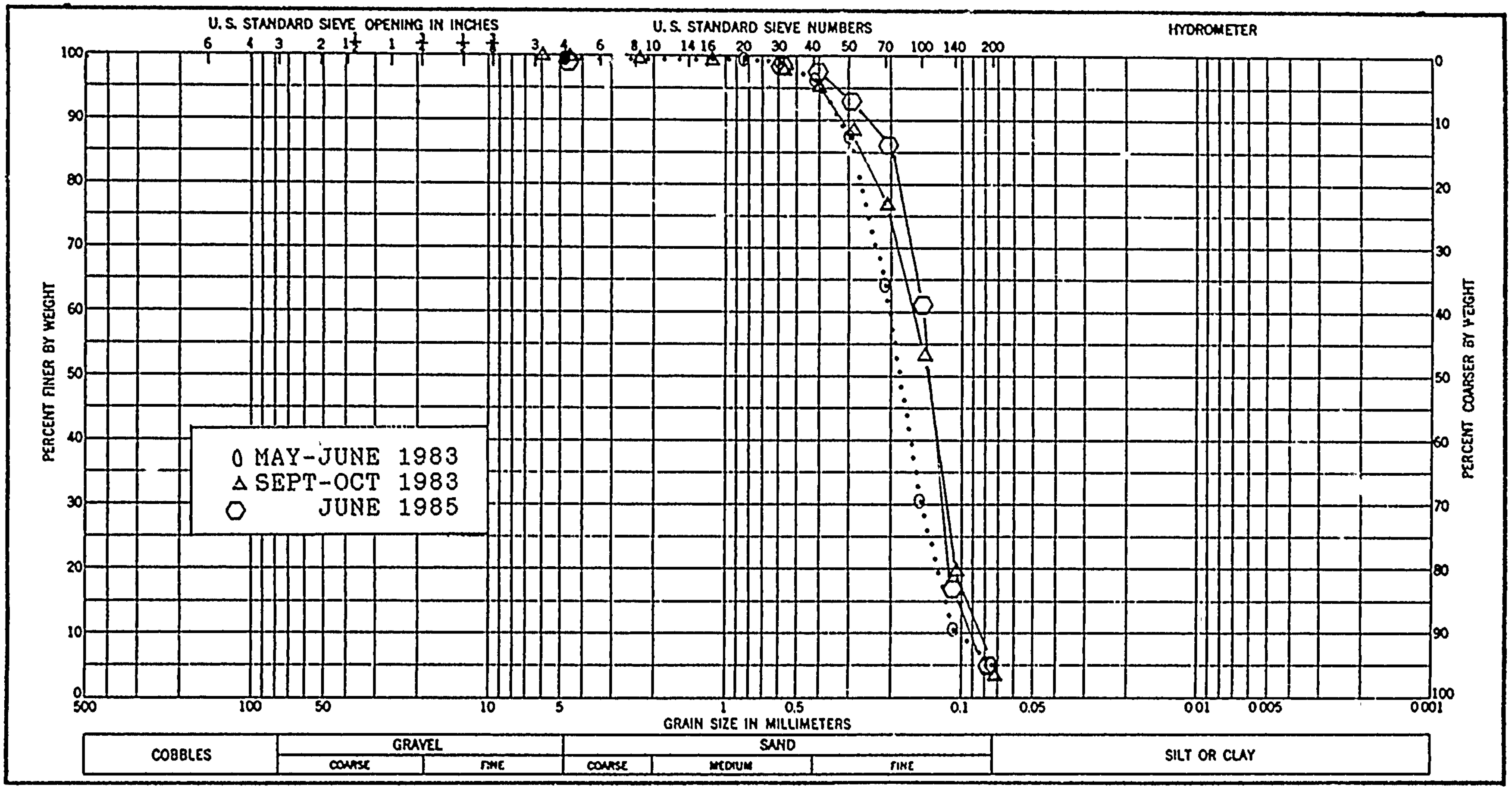


Figure 57. Sediment gradation curves for Sta 495200

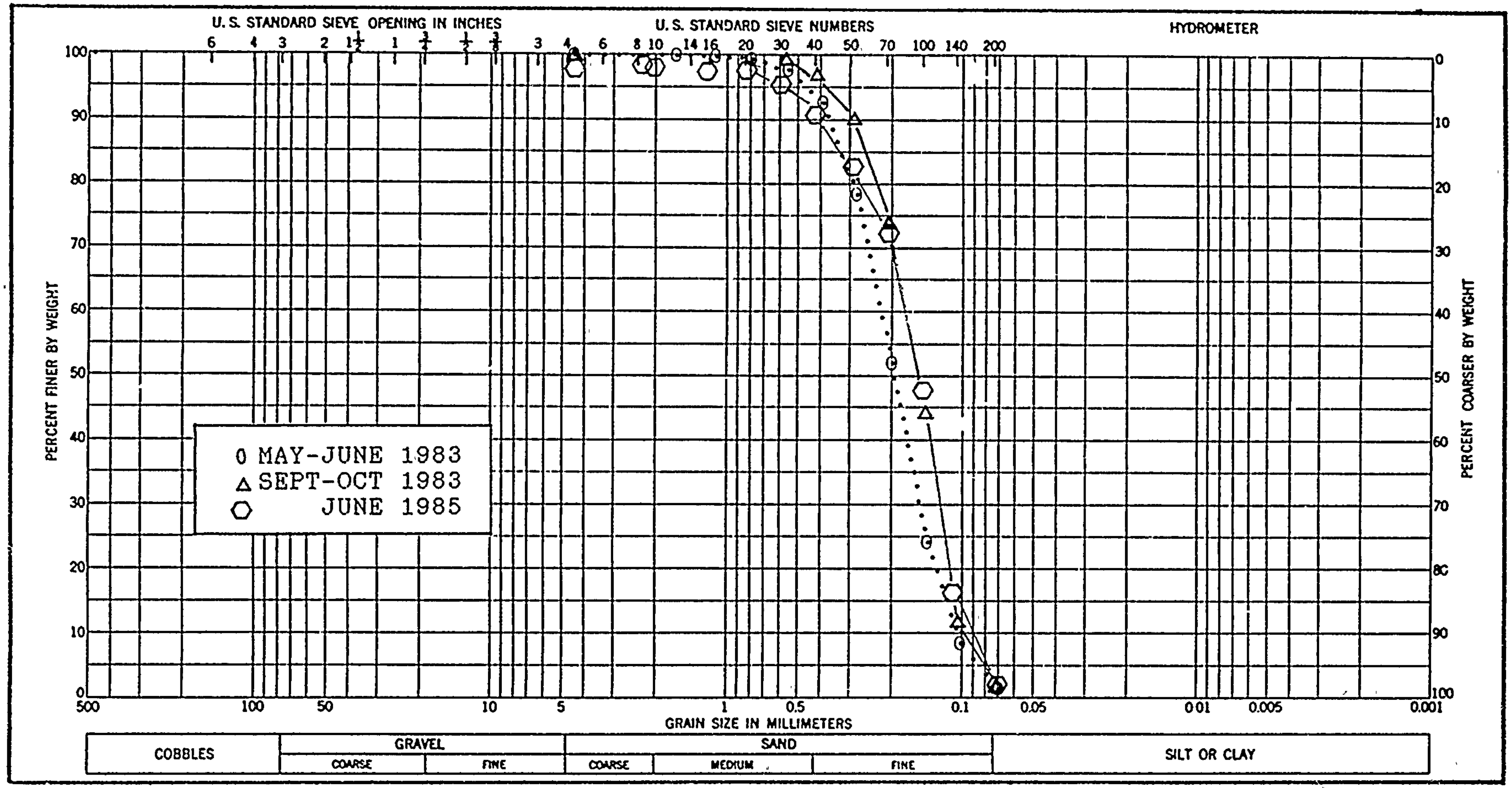


Figure 58. Sediment gradation curves for Sta 495400

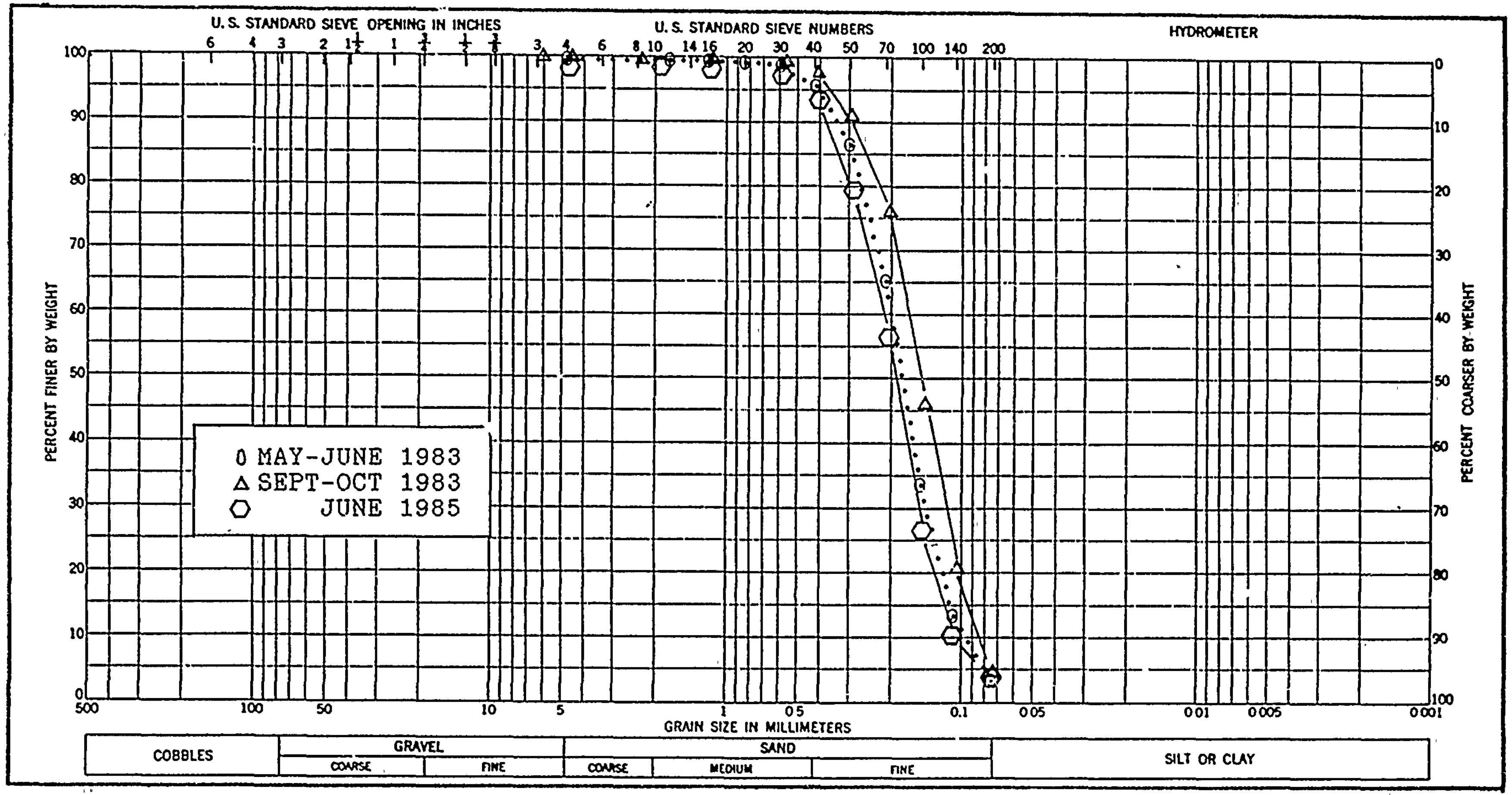


Figure 59. Sediment gradation curves for Sta 495700

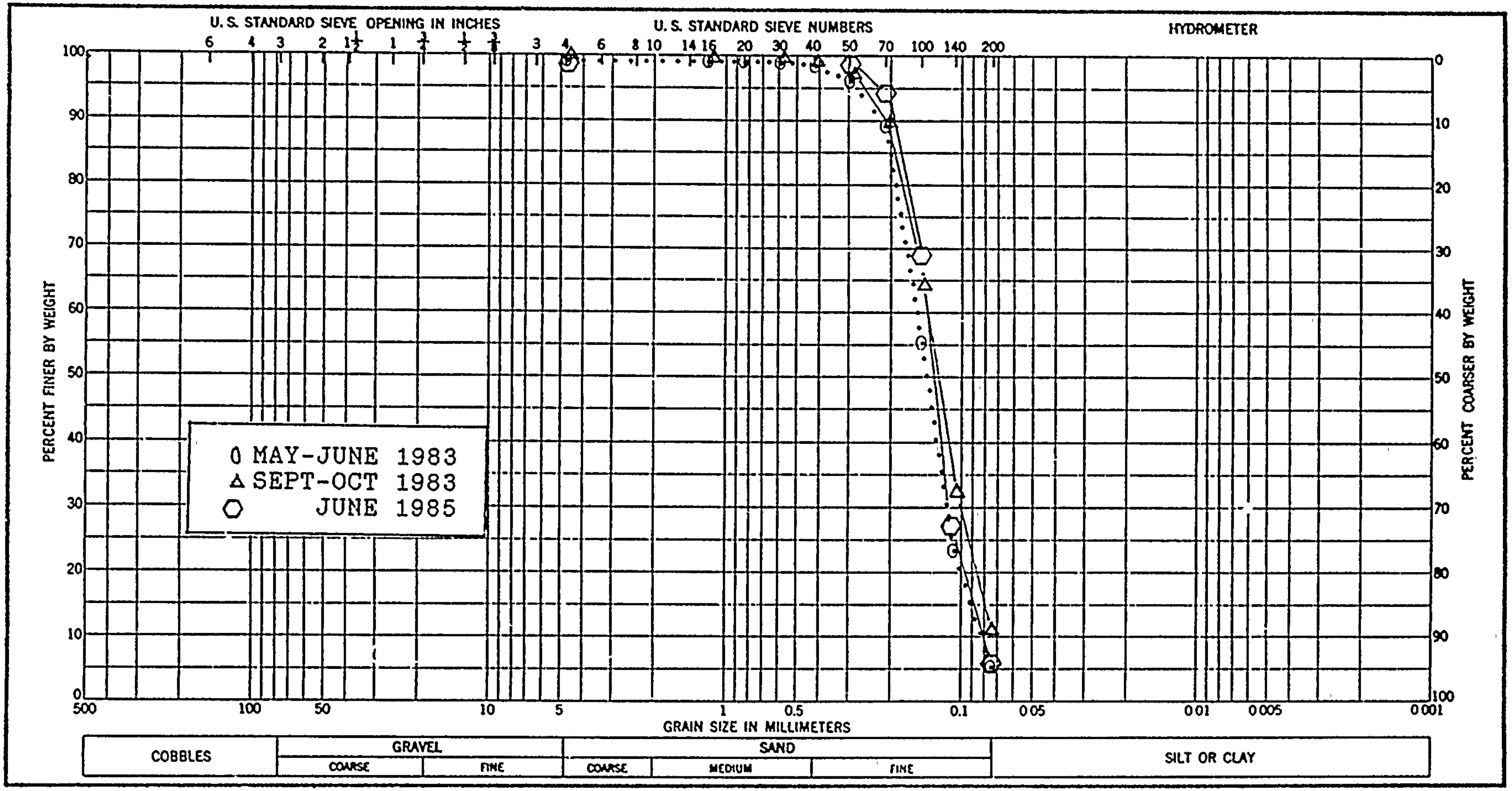


Figure 60. Sediment gradation curves for Sta 4951000

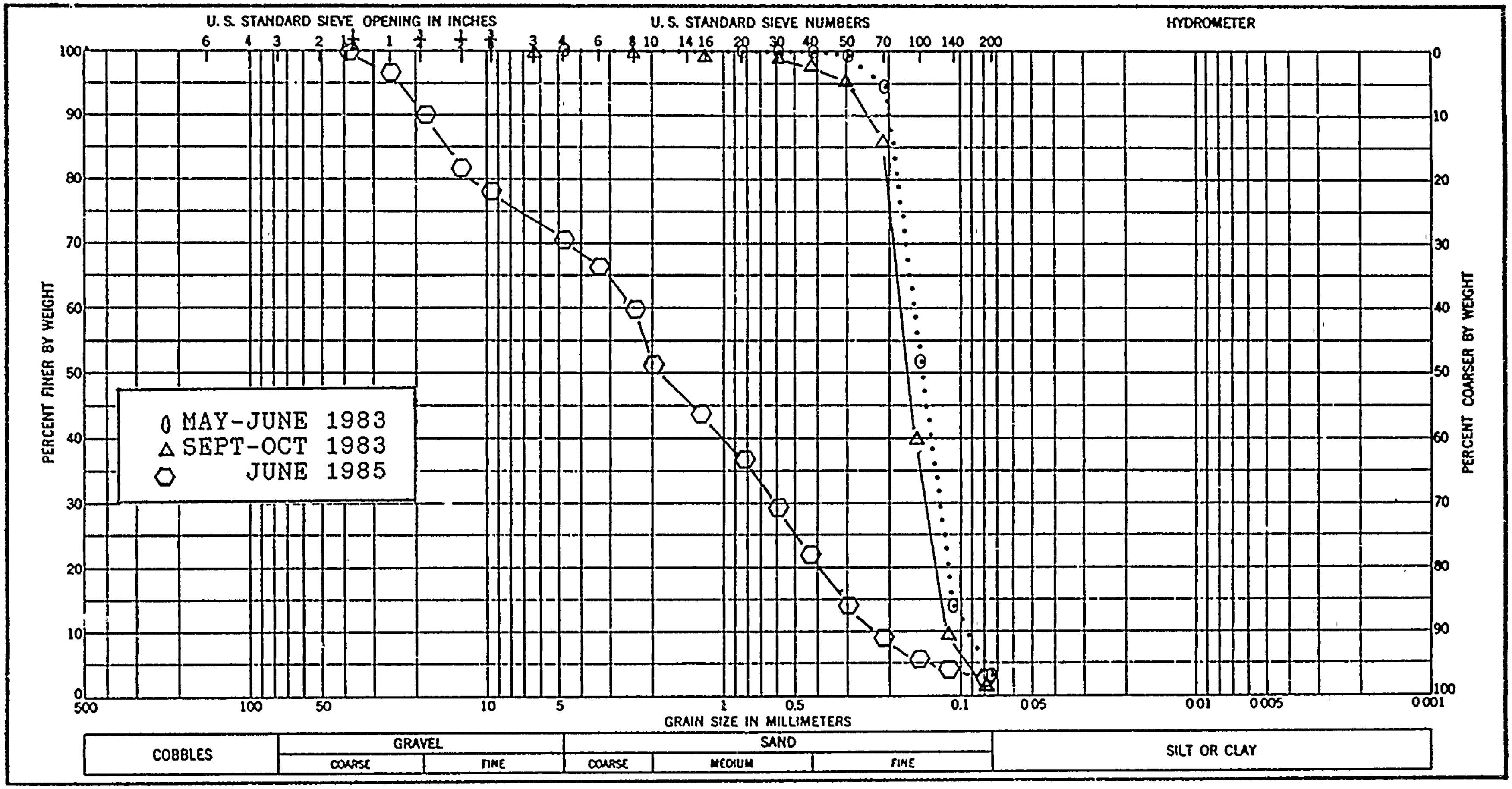


Figure 61. Sediment gradation curves for Sta 501700

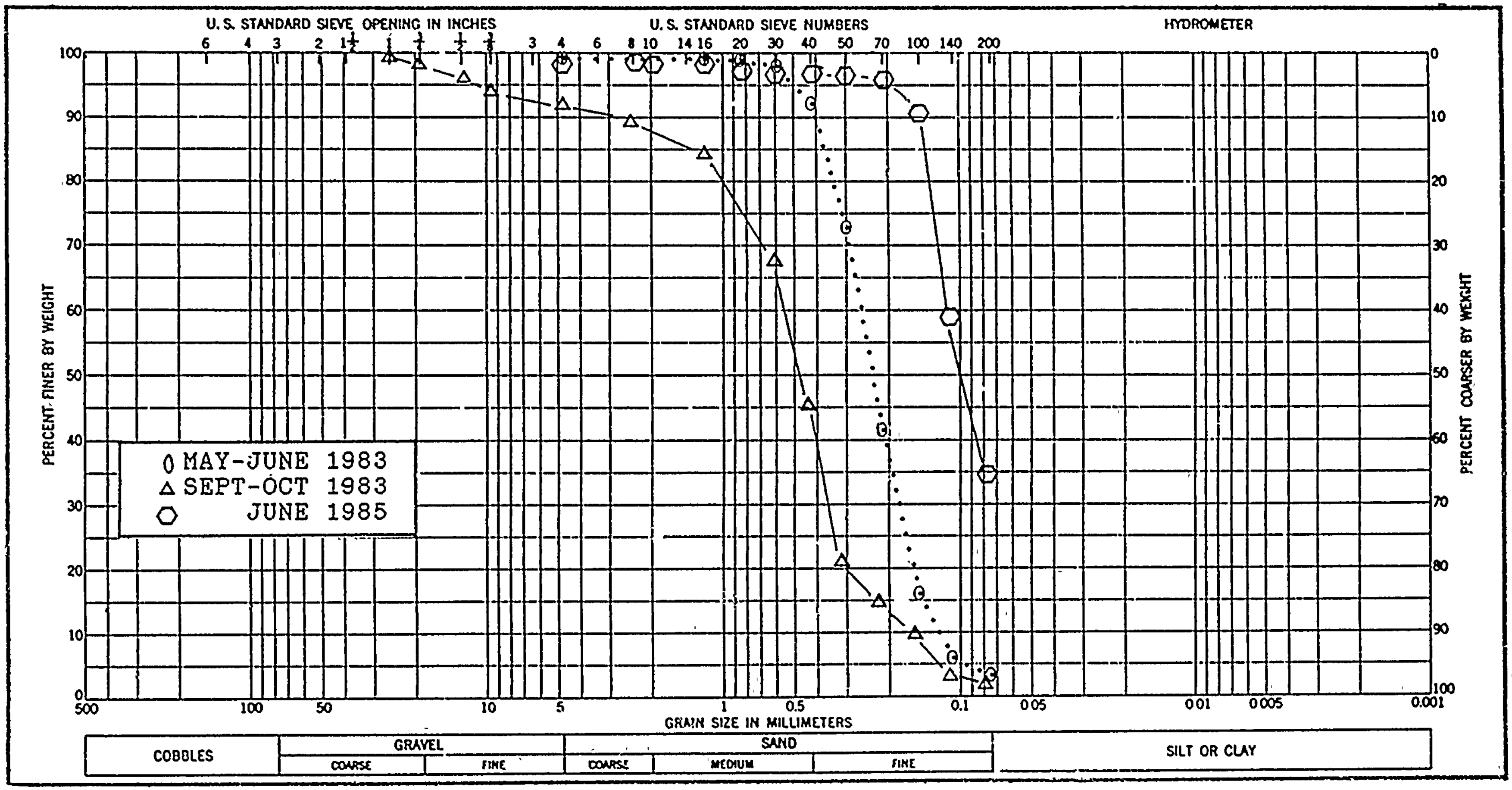


Figure 62. Sediment gradation curves for Sta 501900

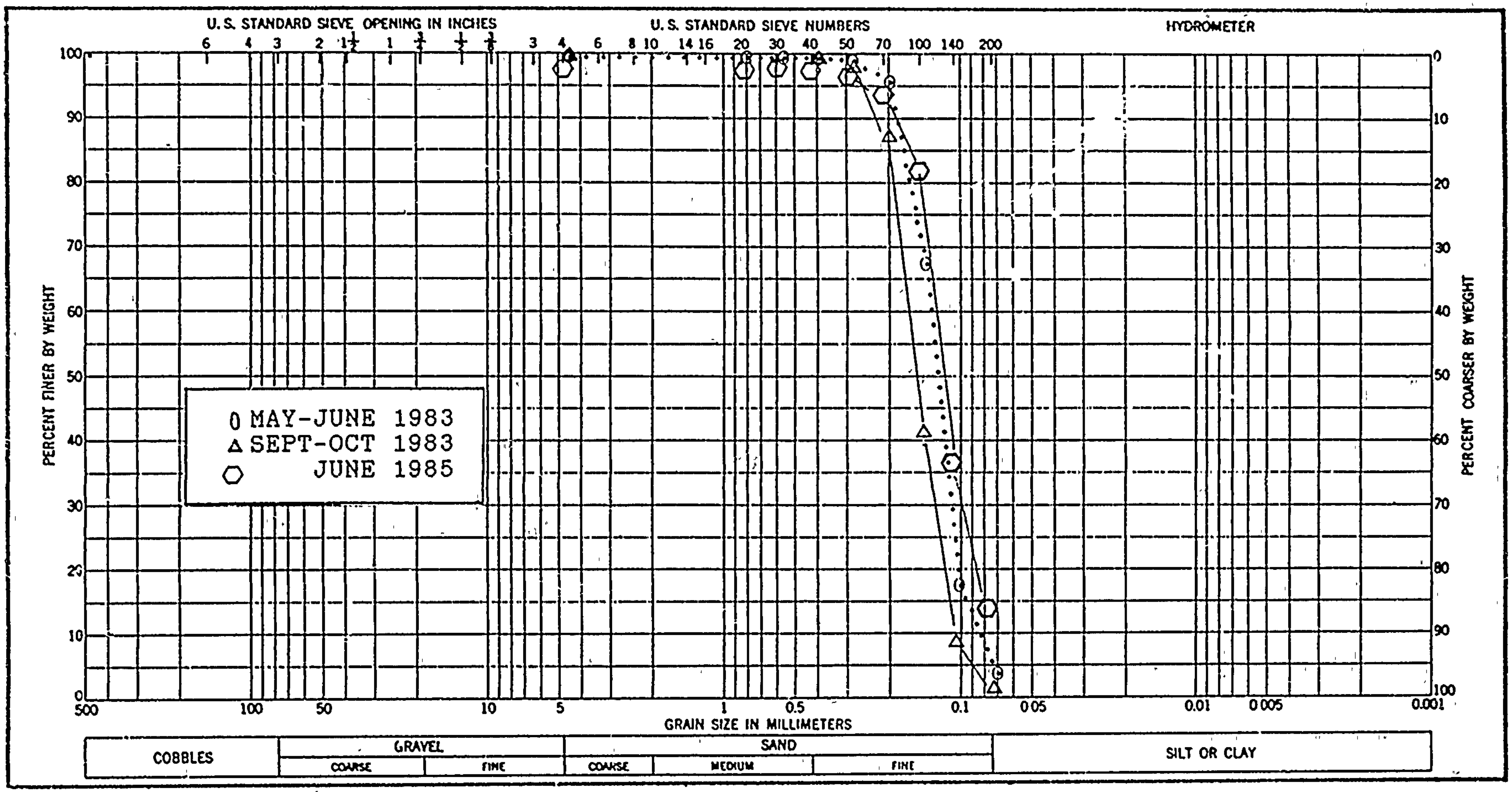


Figure 63. Sediment gradation curves for Sta 5011200

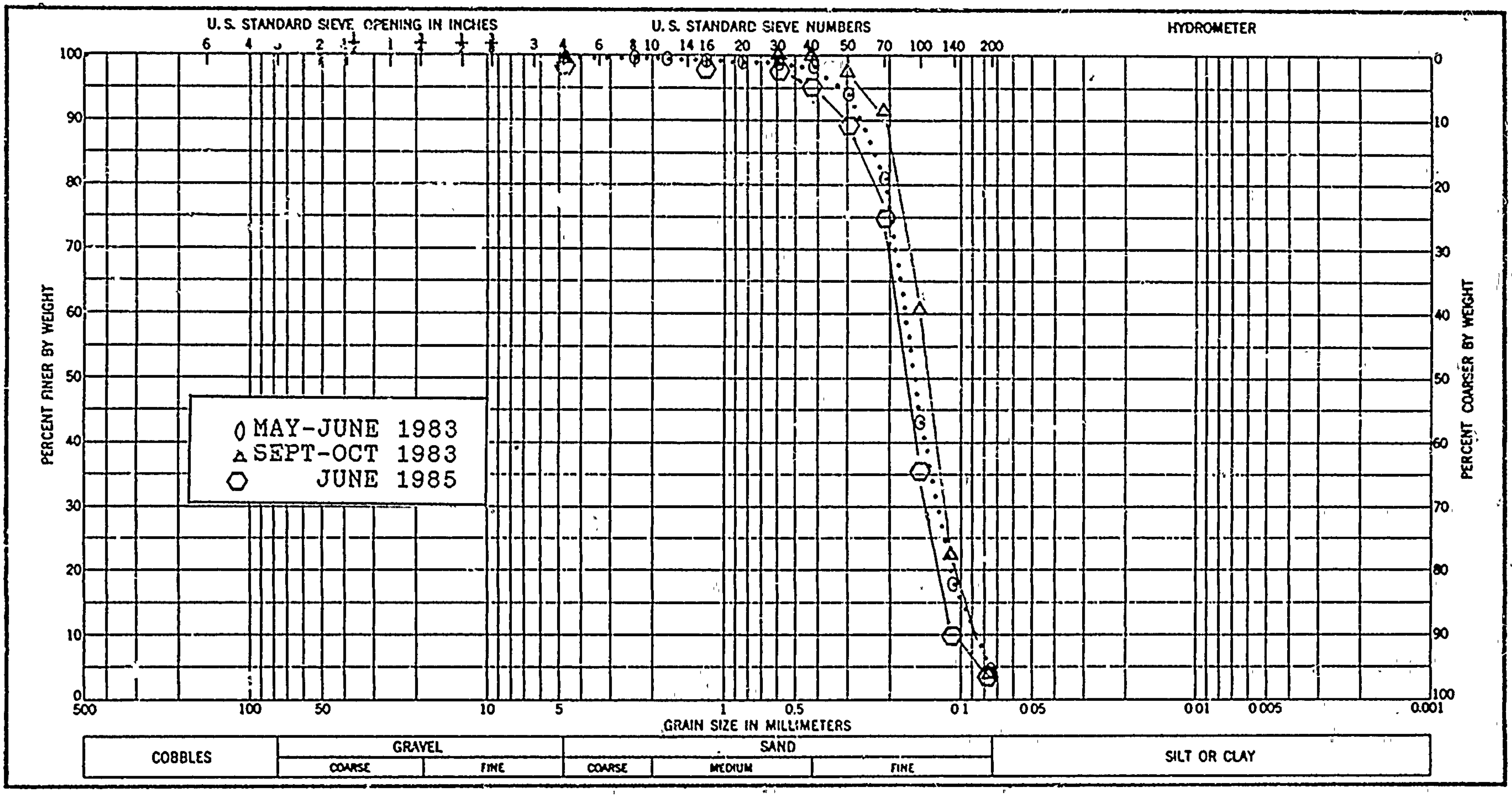


Figure 64. Sediment gradation curves for Sta 510300

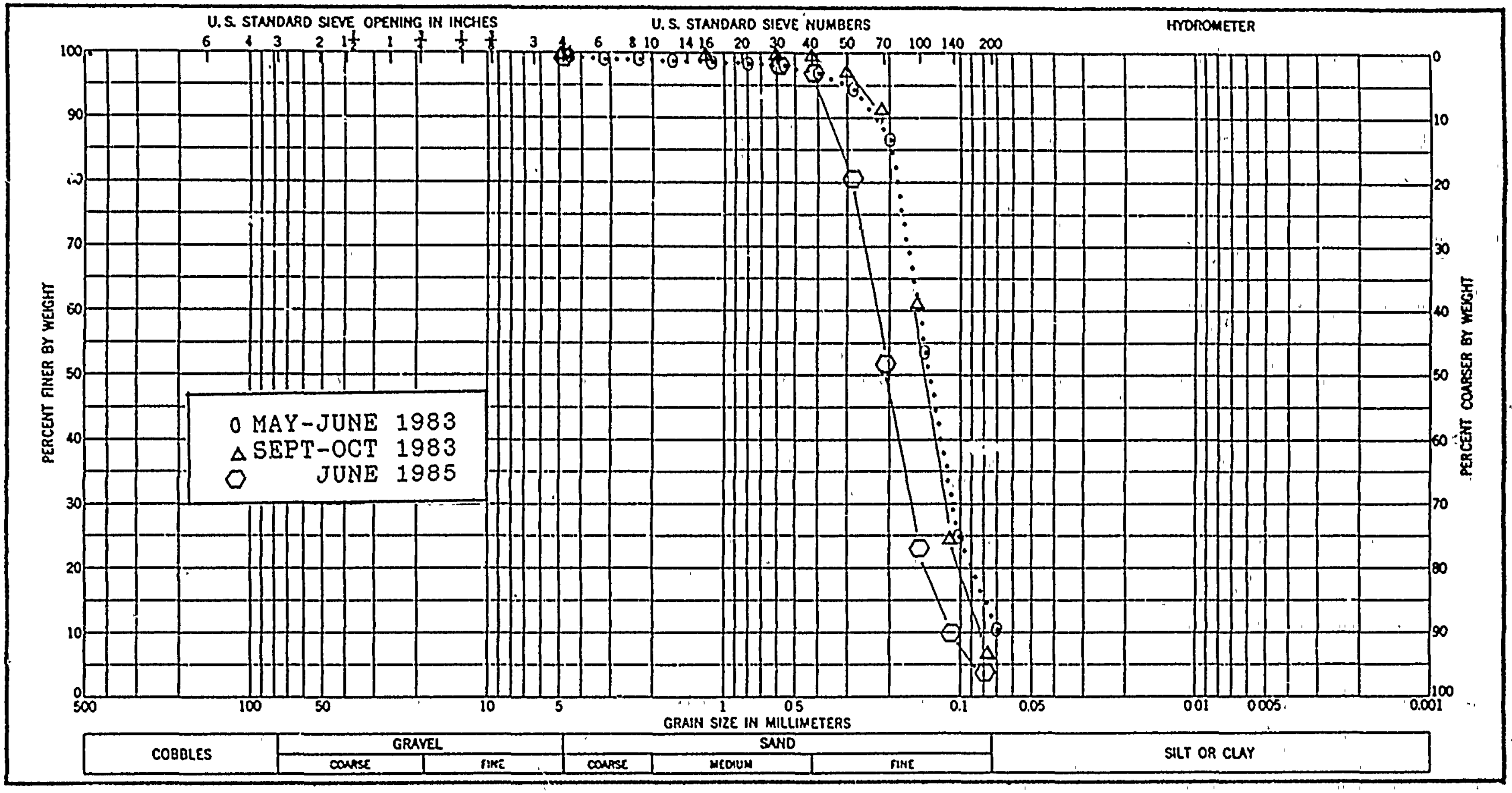


Figure 65. Sediment gradation curves for Sta 510800

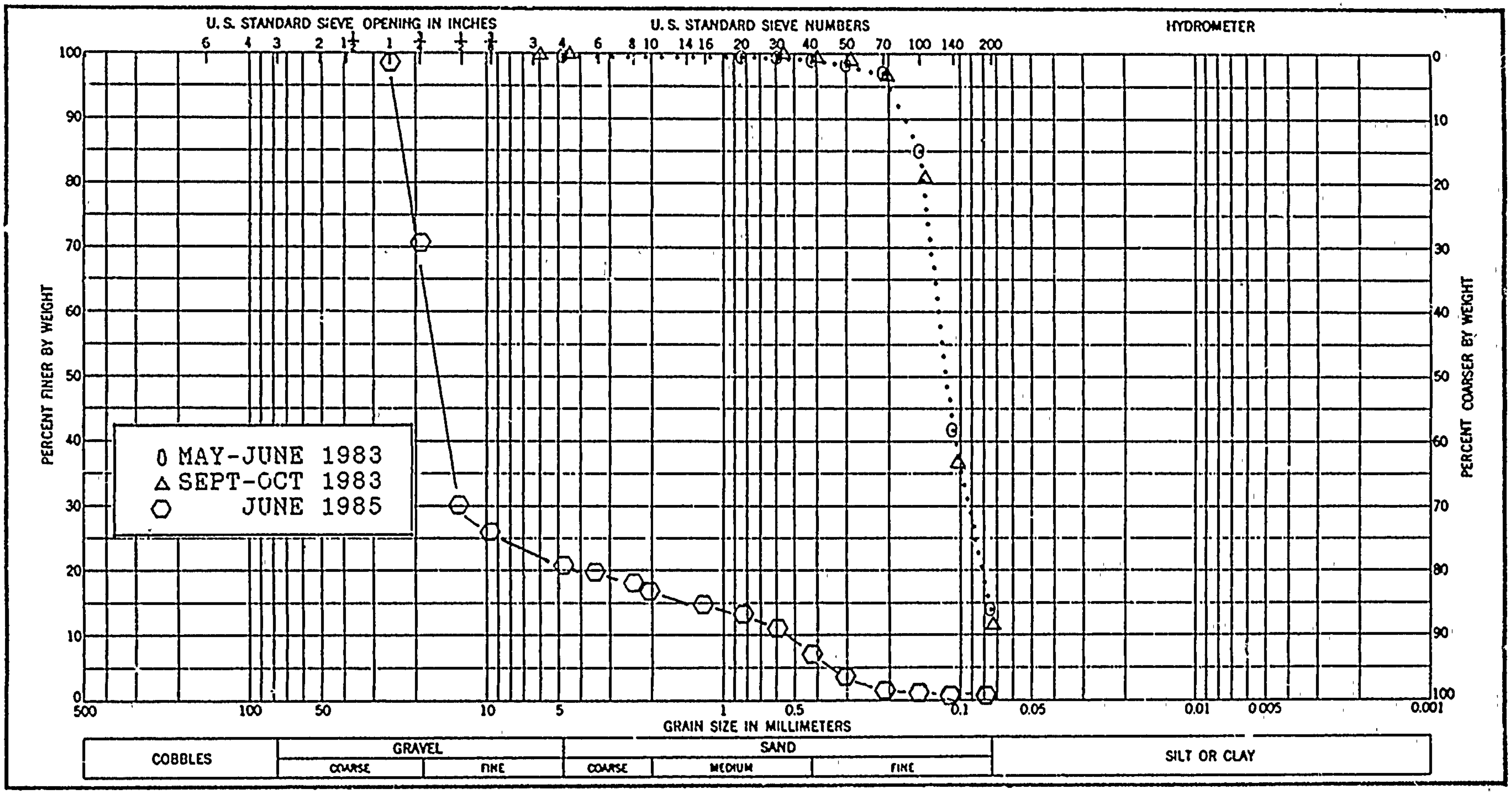
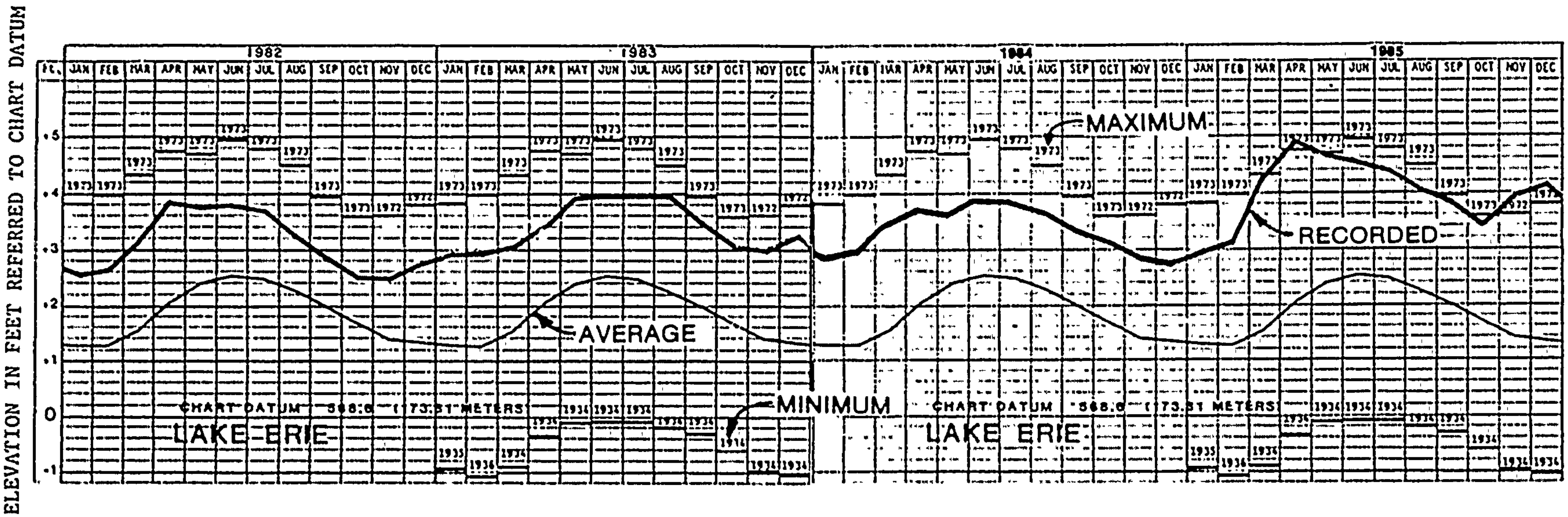


Figure 66. Sediment gradation curves for Sta 5101300



NOTE: The maximum, minimum, and average monthly level is reported/computed for the period of record since 1900.

Figure 67. Monthly lake levels during monitoring program period

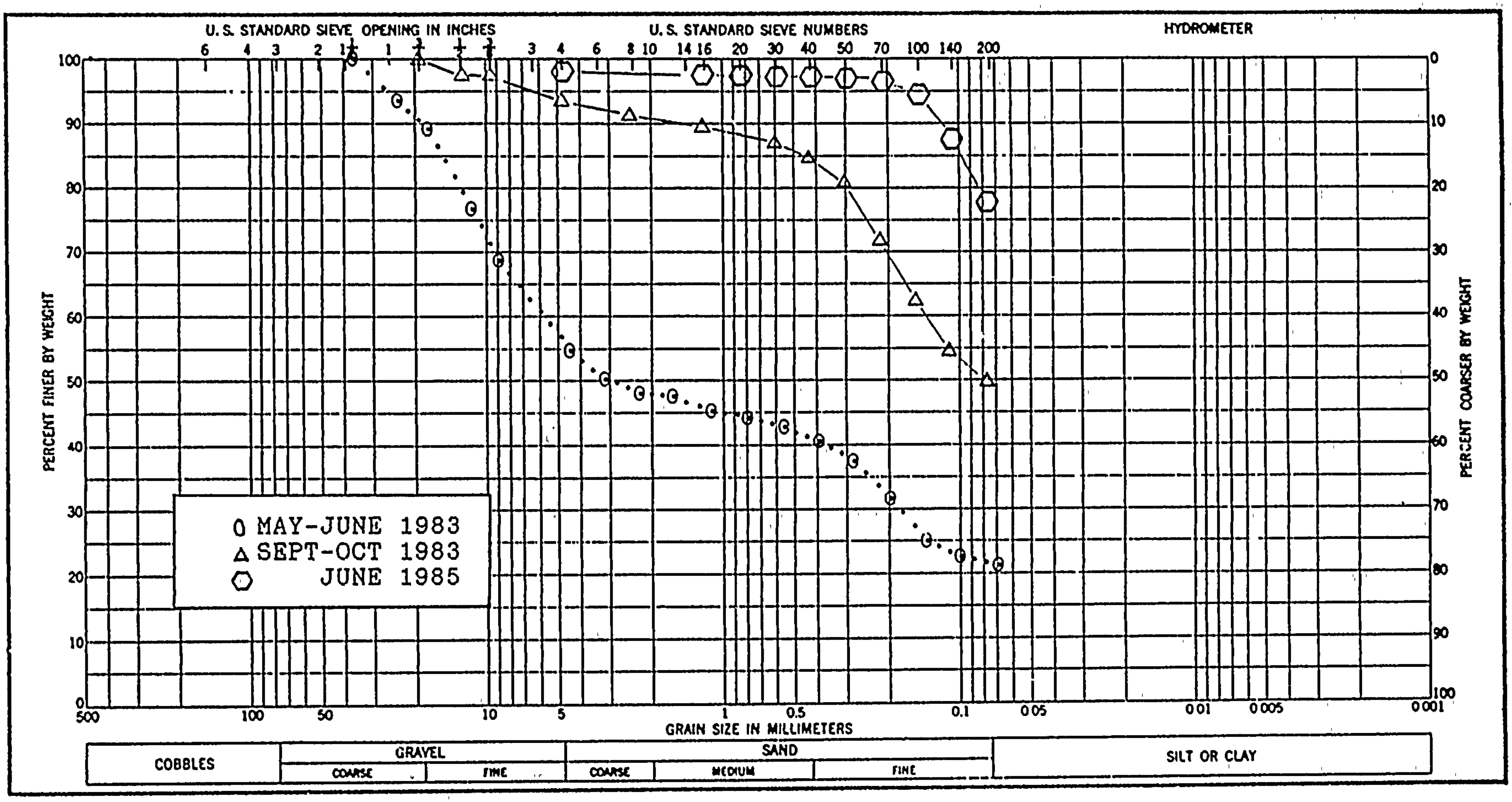


Figure 68. Sediment gradation curves for creek Sta 55220

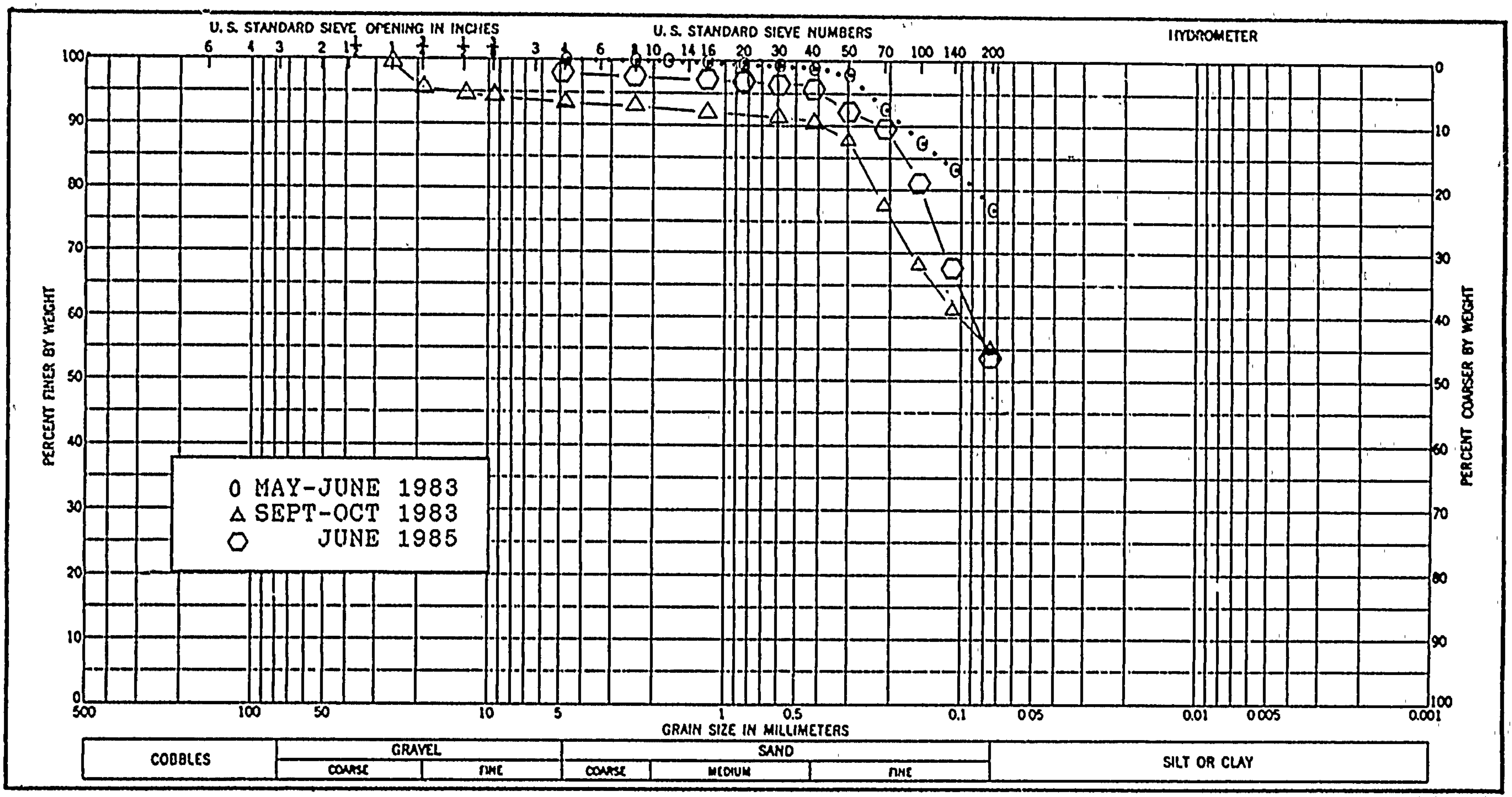


Figure 69. Sediment gradation curves for creek Sta 60250

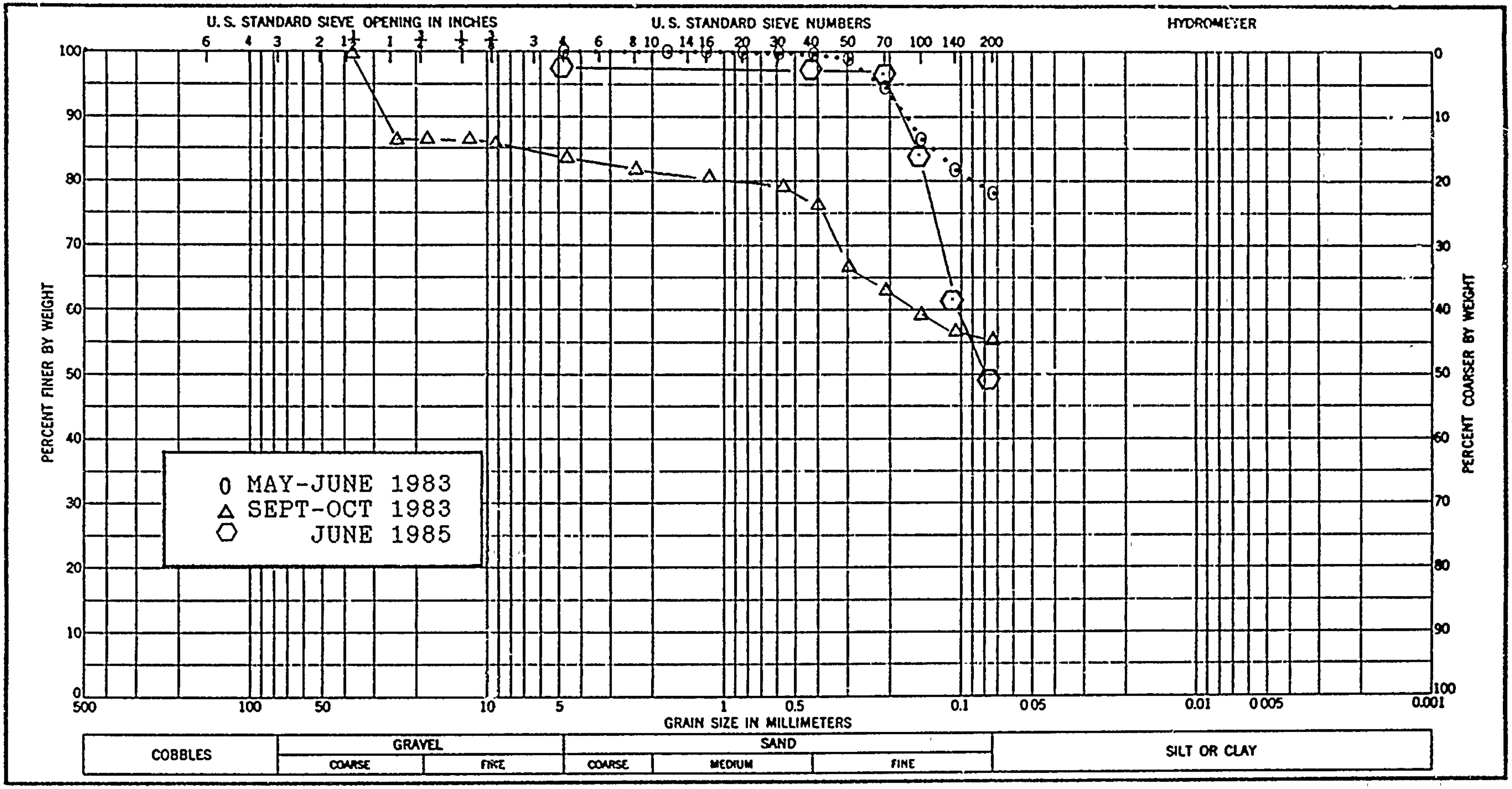


Figure 70. Sediment gradation curves for Creek Sta 65320

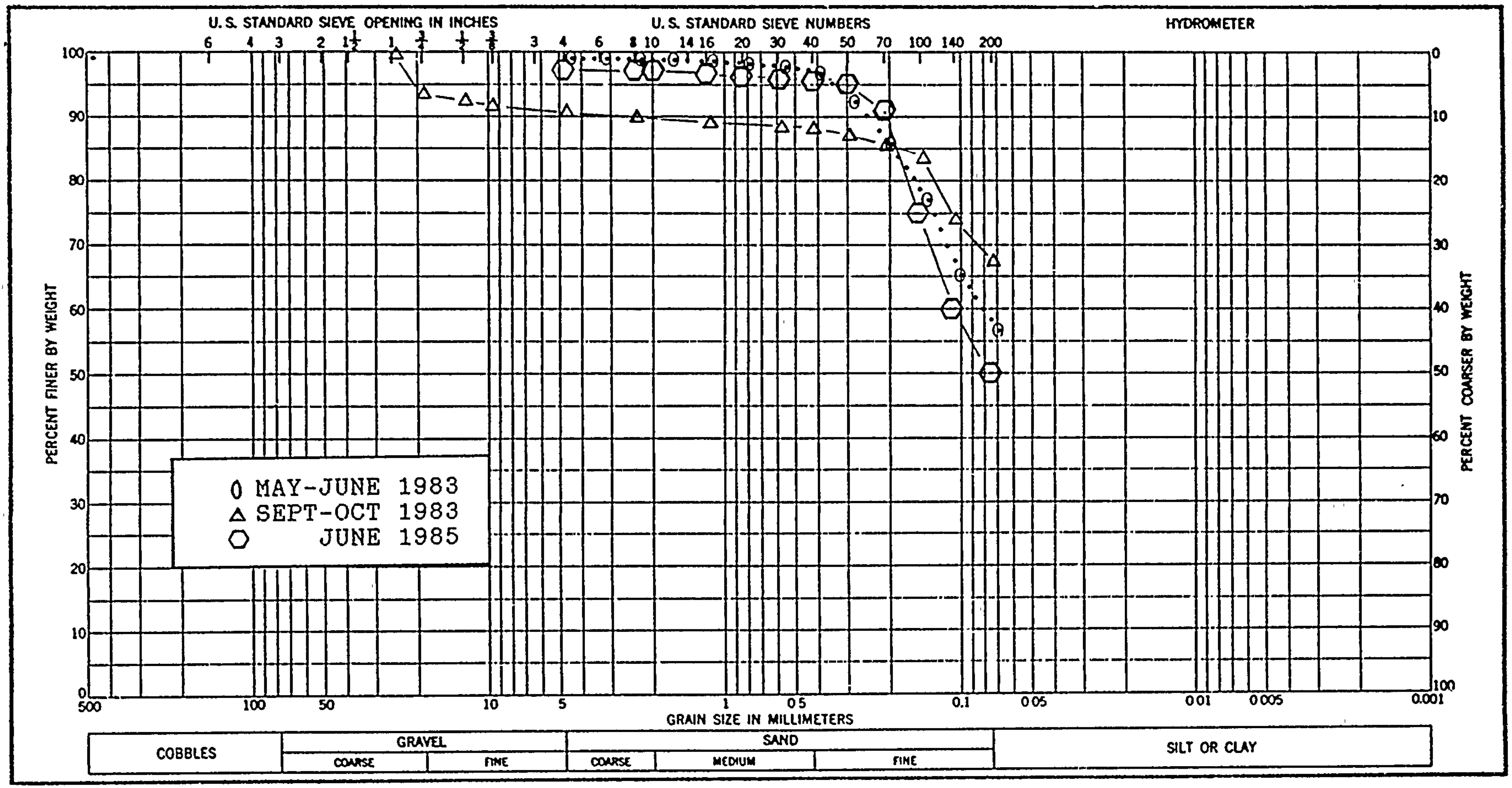


Figure 71. Sediment gradation curves for creek Sta 70230

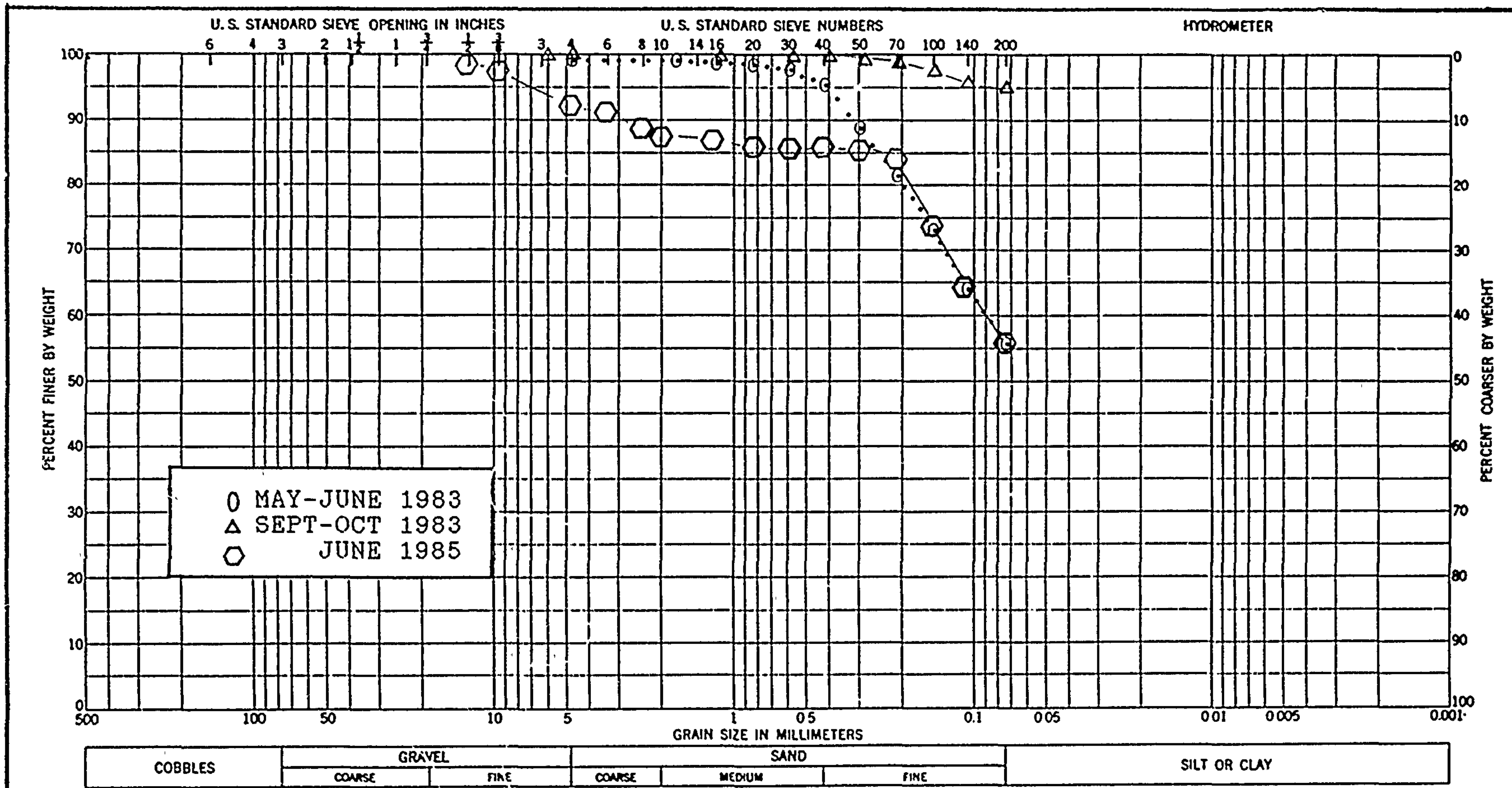


Figure 72. Sediment gradation curves for creek Sta 75160

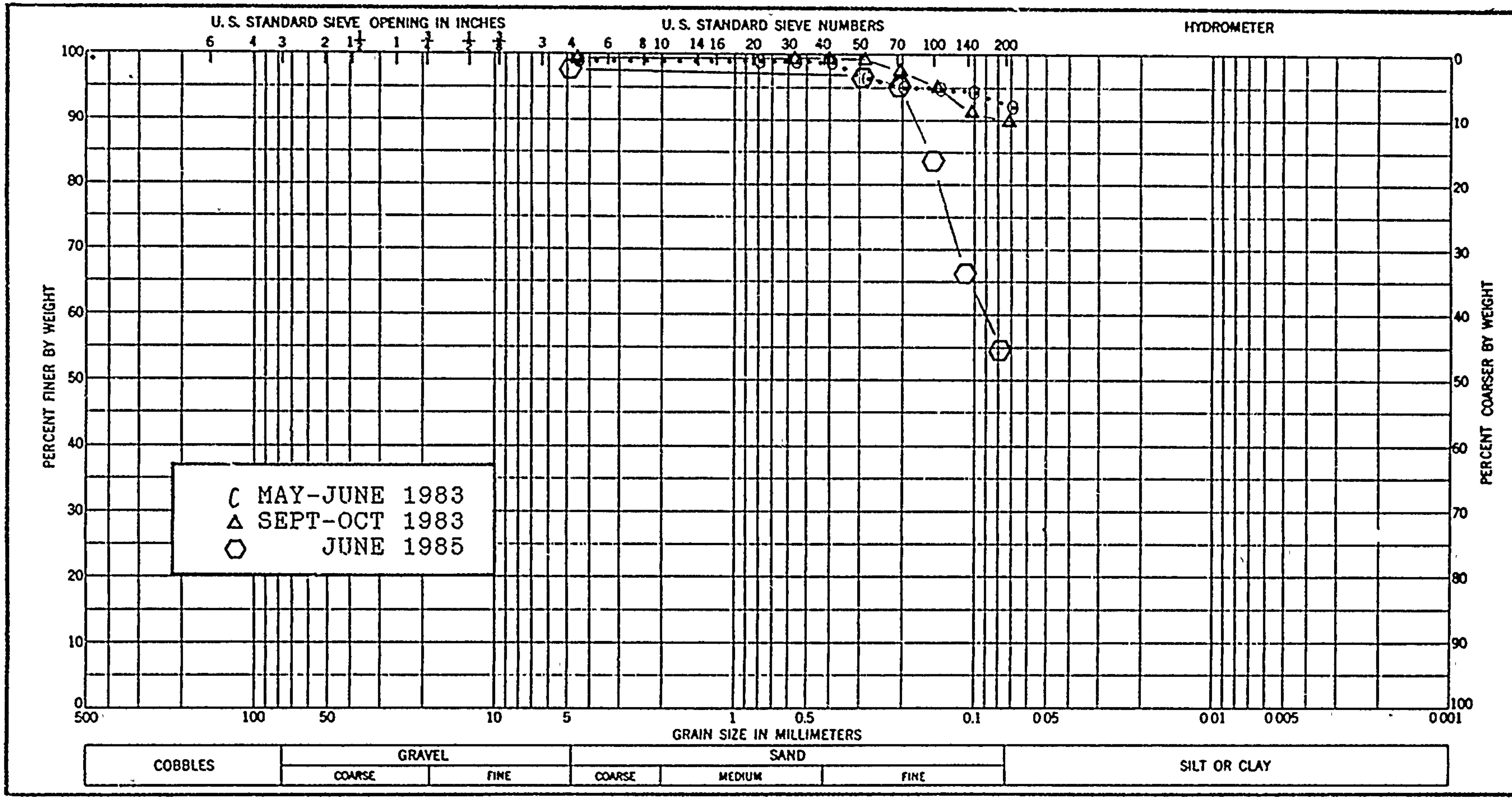


Figure 73. Sediment gradation curves for creek Sta 87200

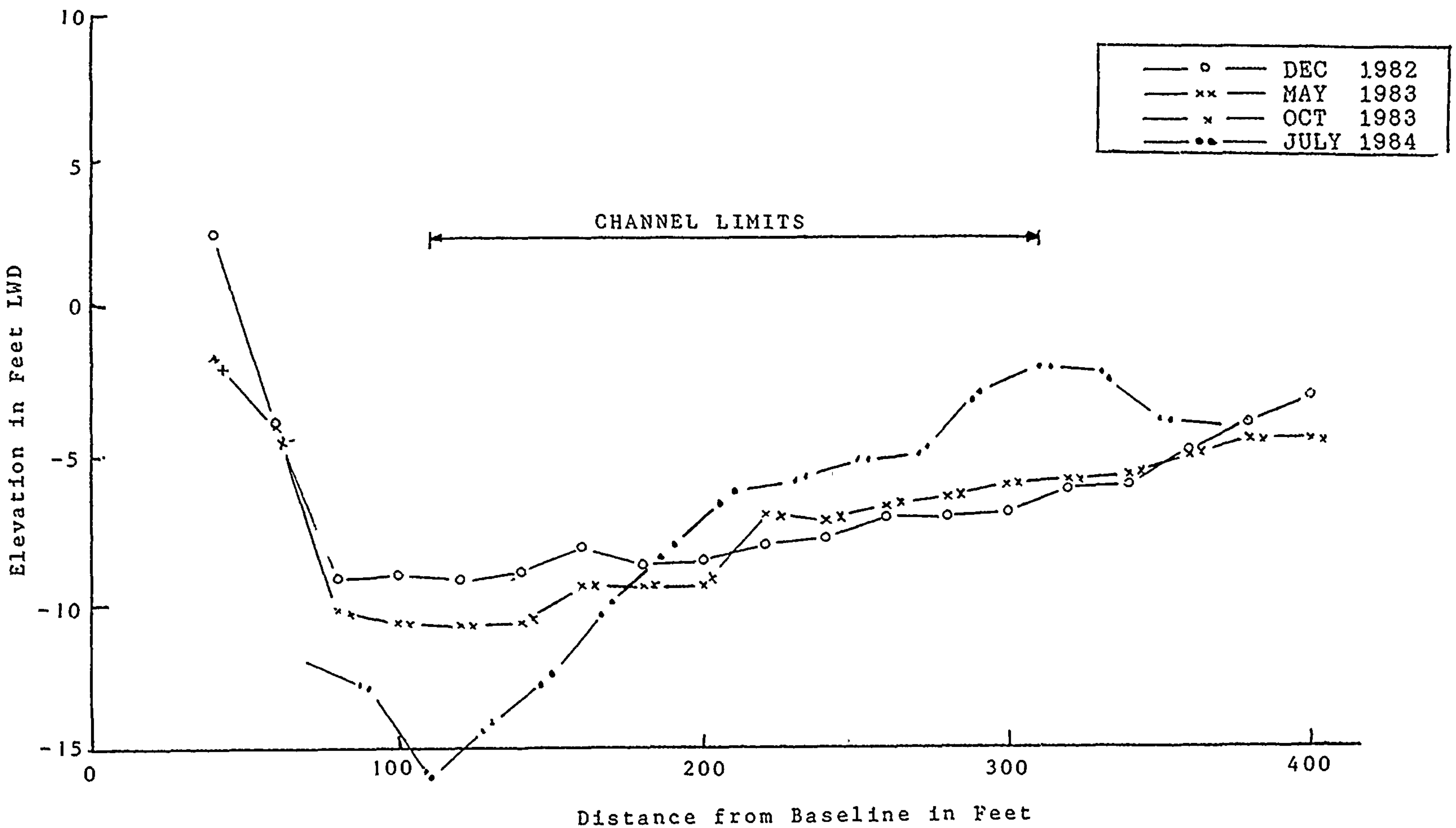


Figure 74. Channel cross sections for Sta 50+00, 1982-1984

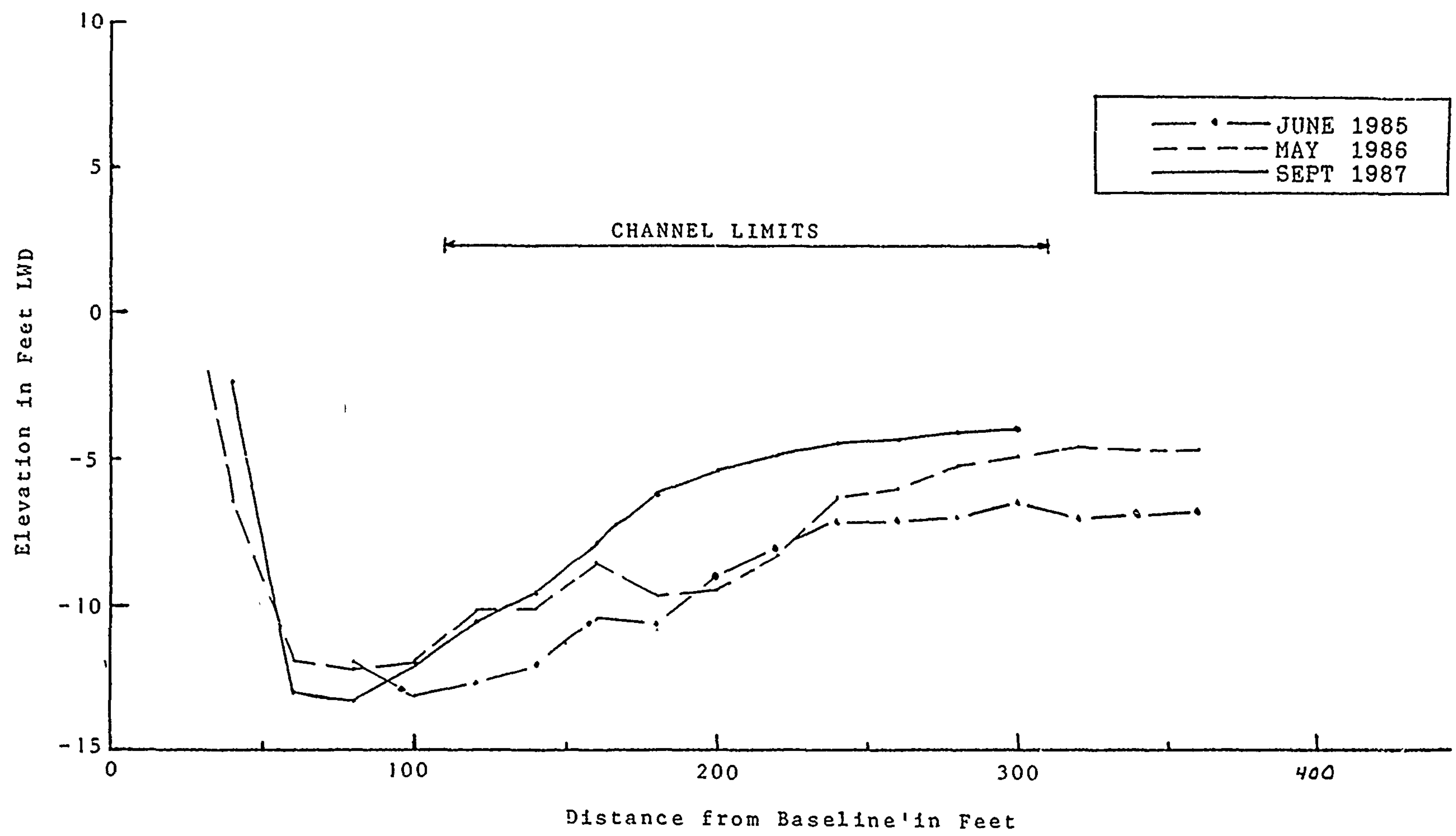


Figure 75. Channel cross sections for Sta 50+00, 1985-1987

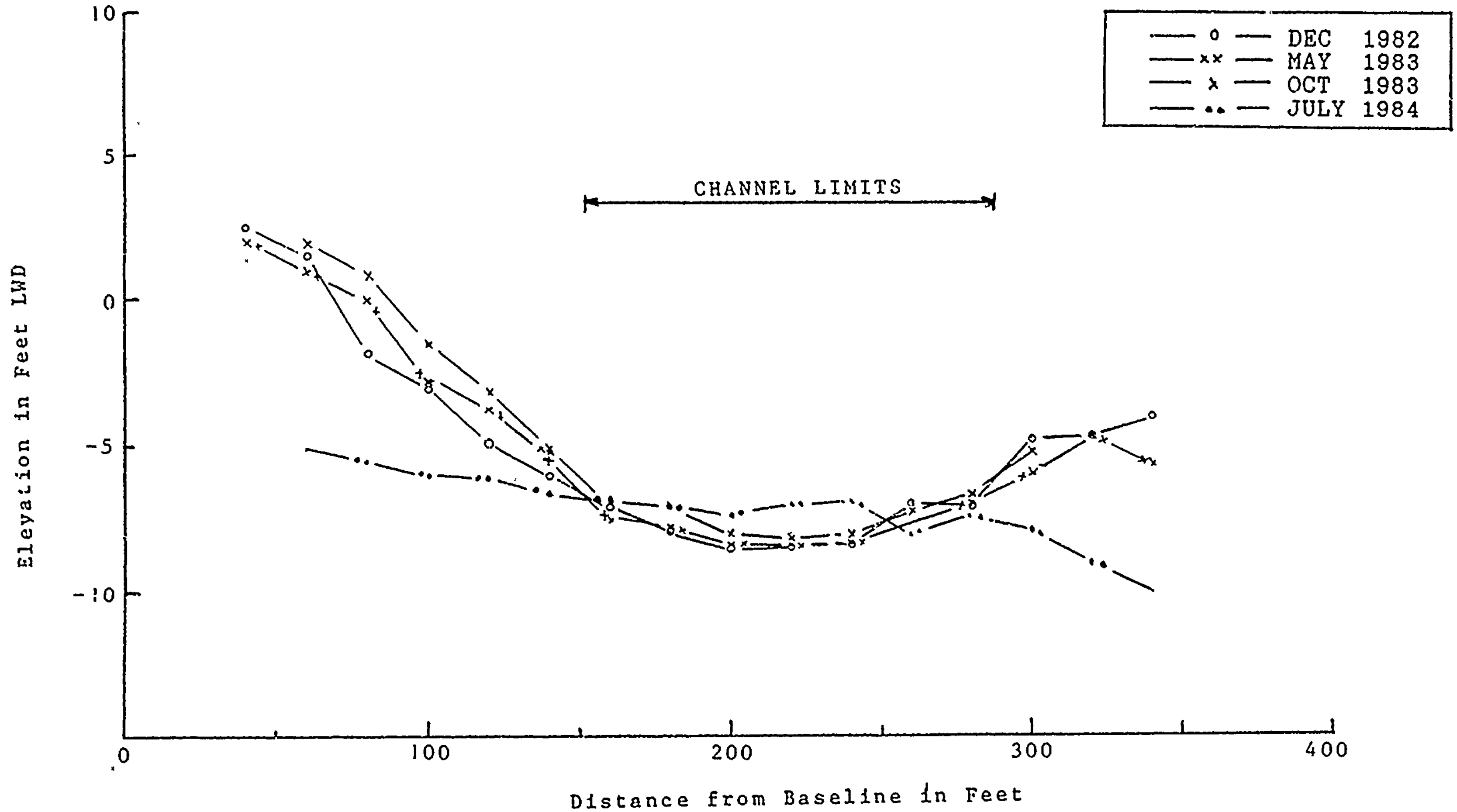


Figure 76. Channel cross sections for Sta 55+00, 1982-1984

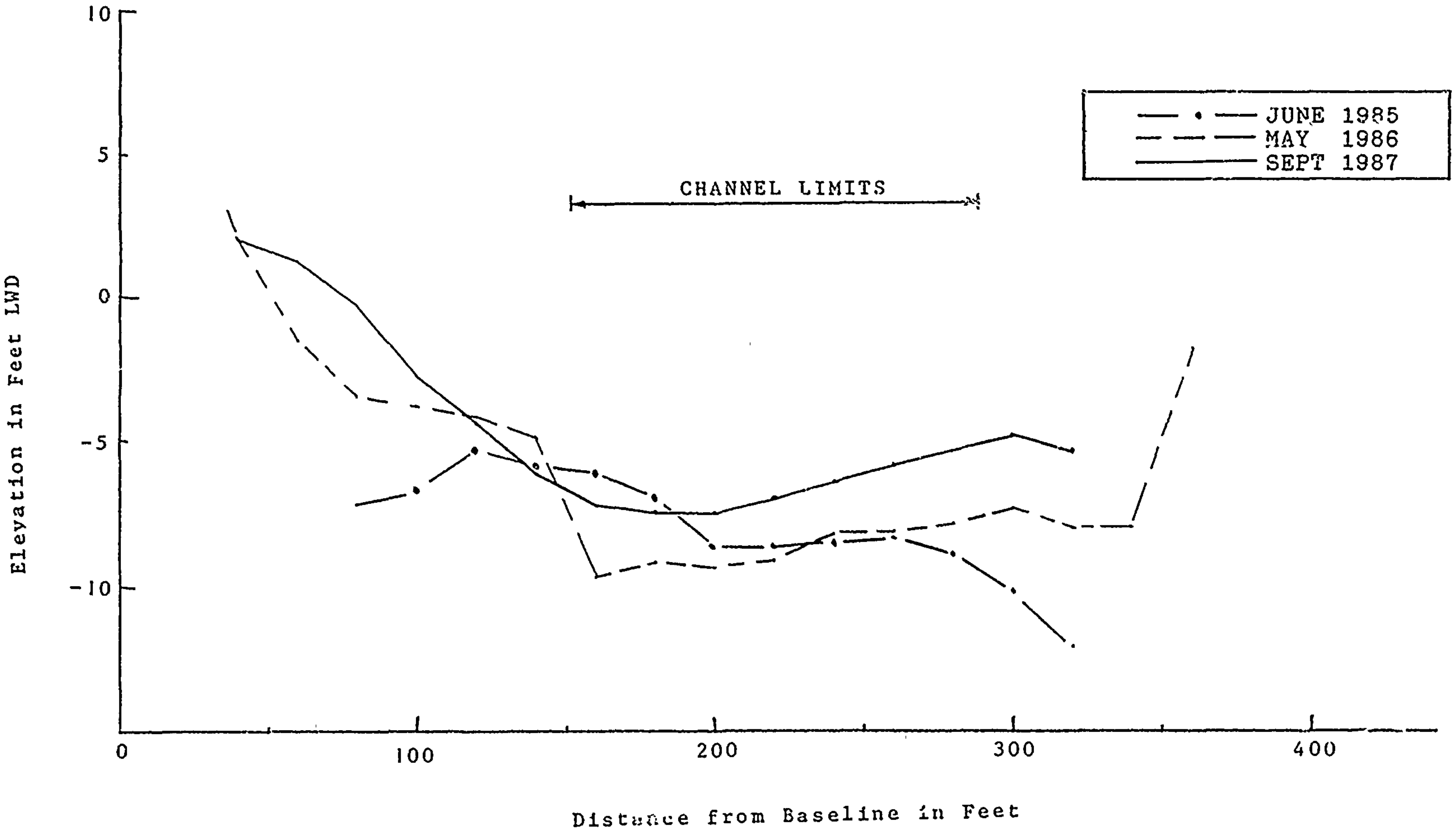


Figure 77. Channel cross sections for Sta 55+00. 1985-1987

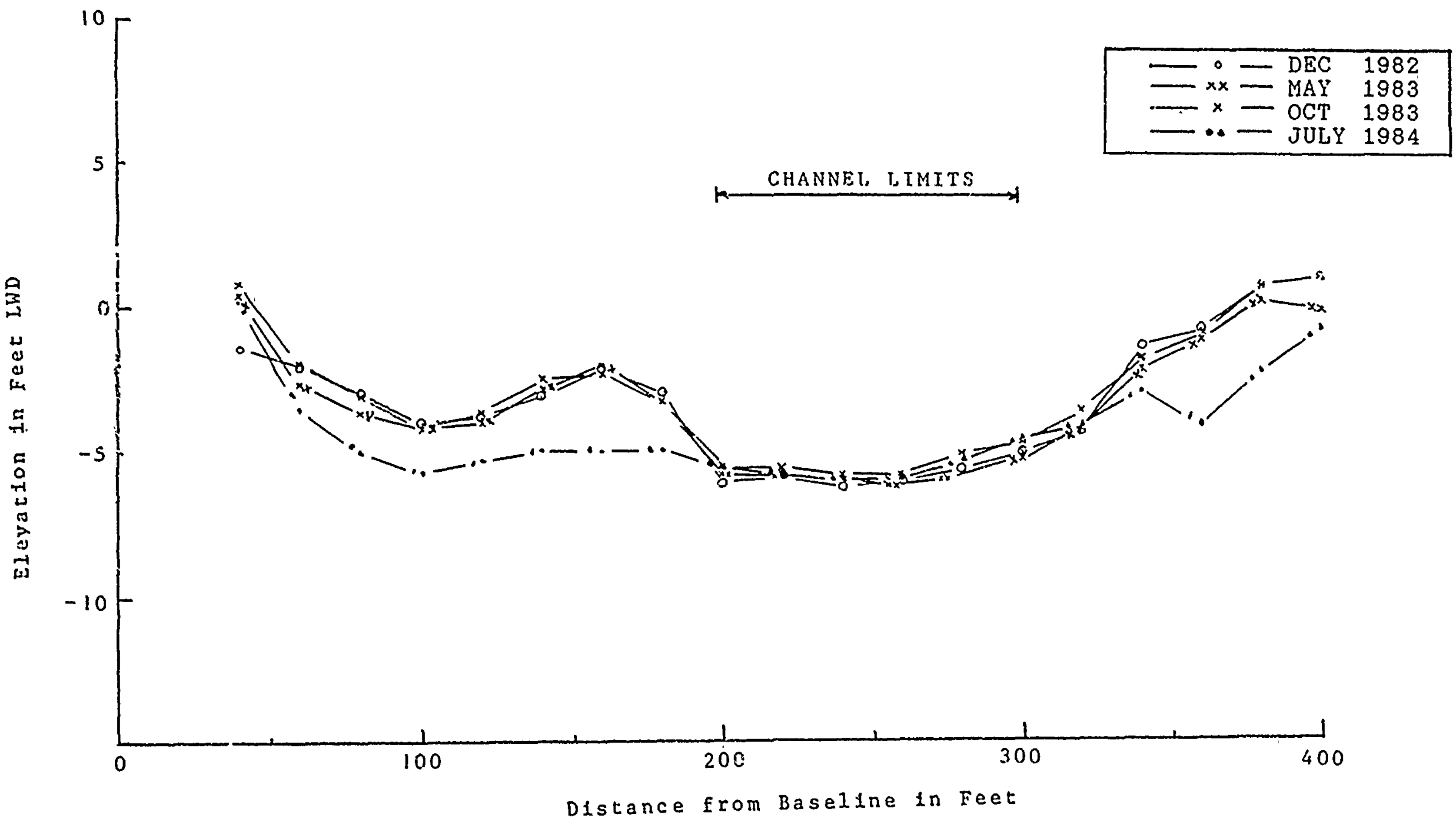


Figure 78. Channel cross sections for Sta 60+00, 1982-1984

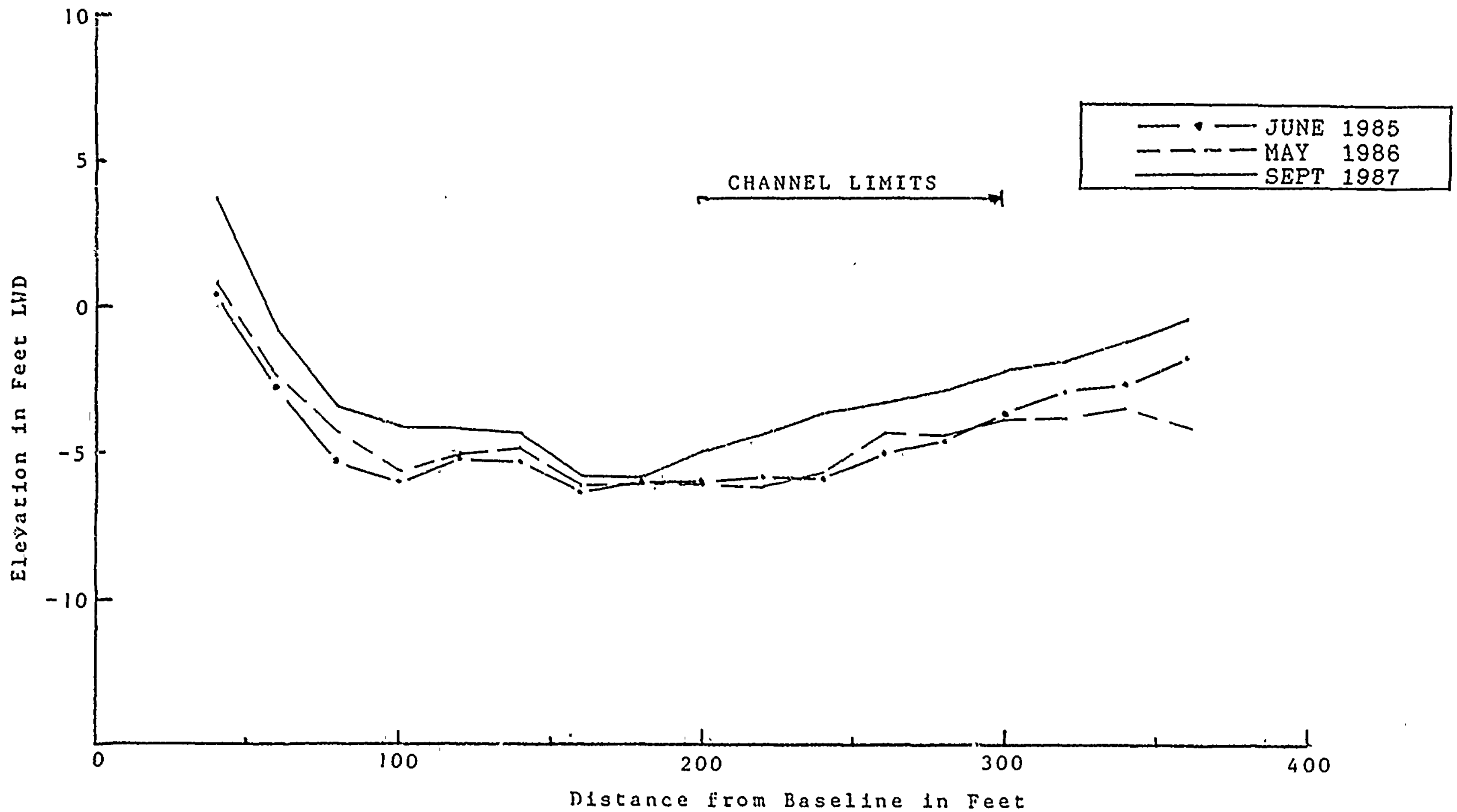


Figure 79. Channel cross sections for Sta 60+00, 1985-1987

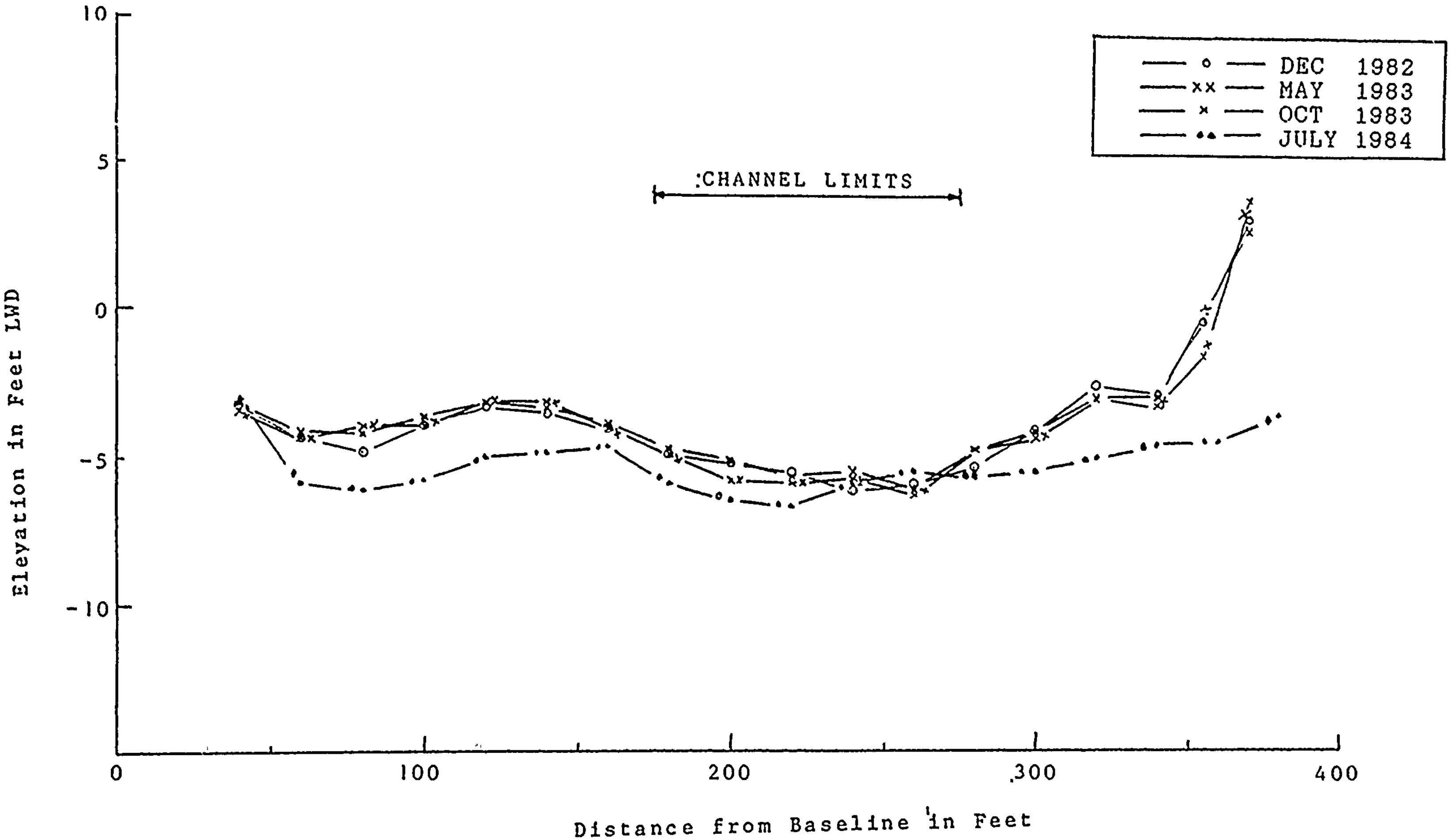


Figure 80. Channel cross sections for Sta 70+00, 1982-1984

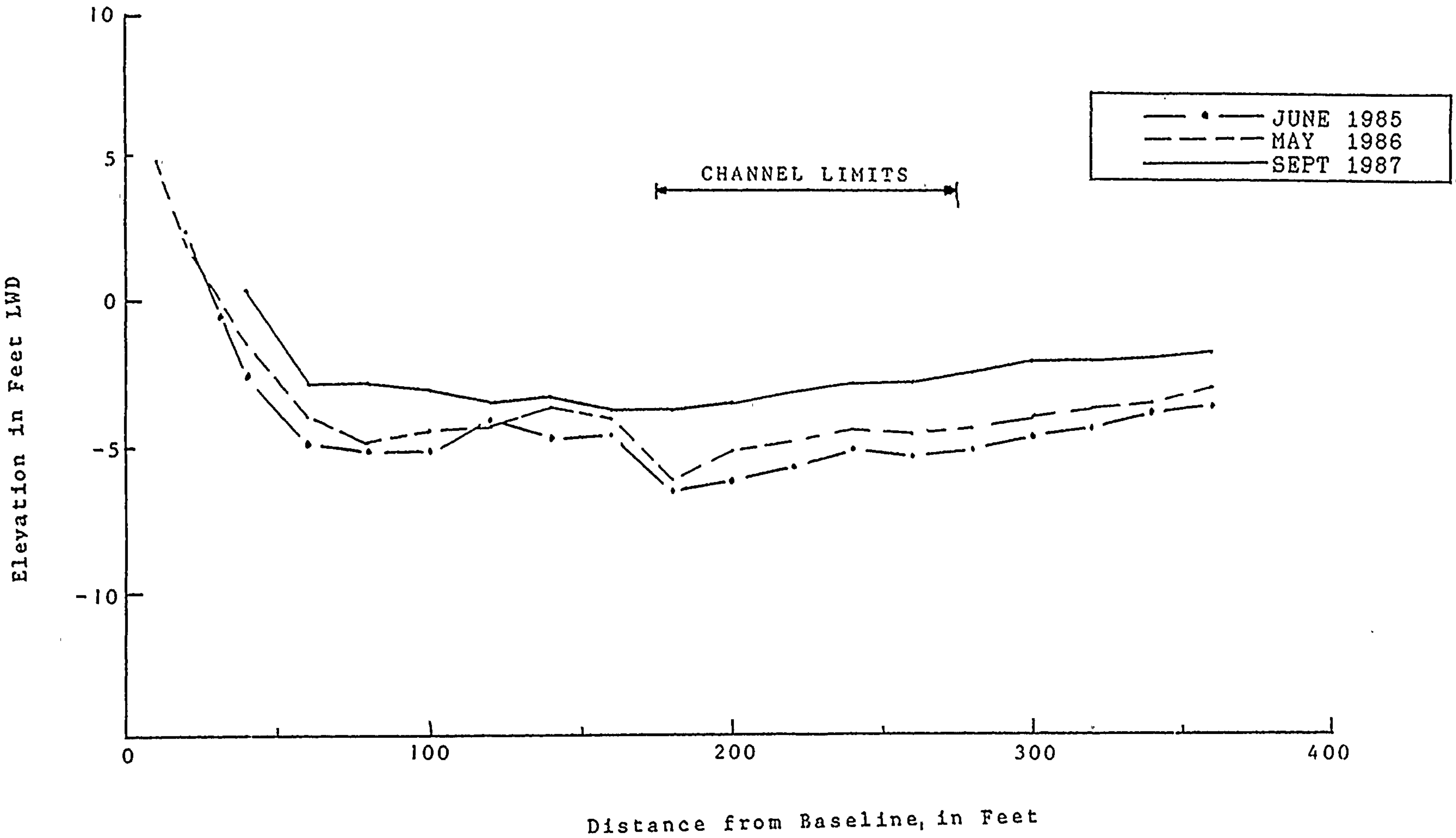


Figure 81. Channel cross sections for Sta 70+00, 1985-1987

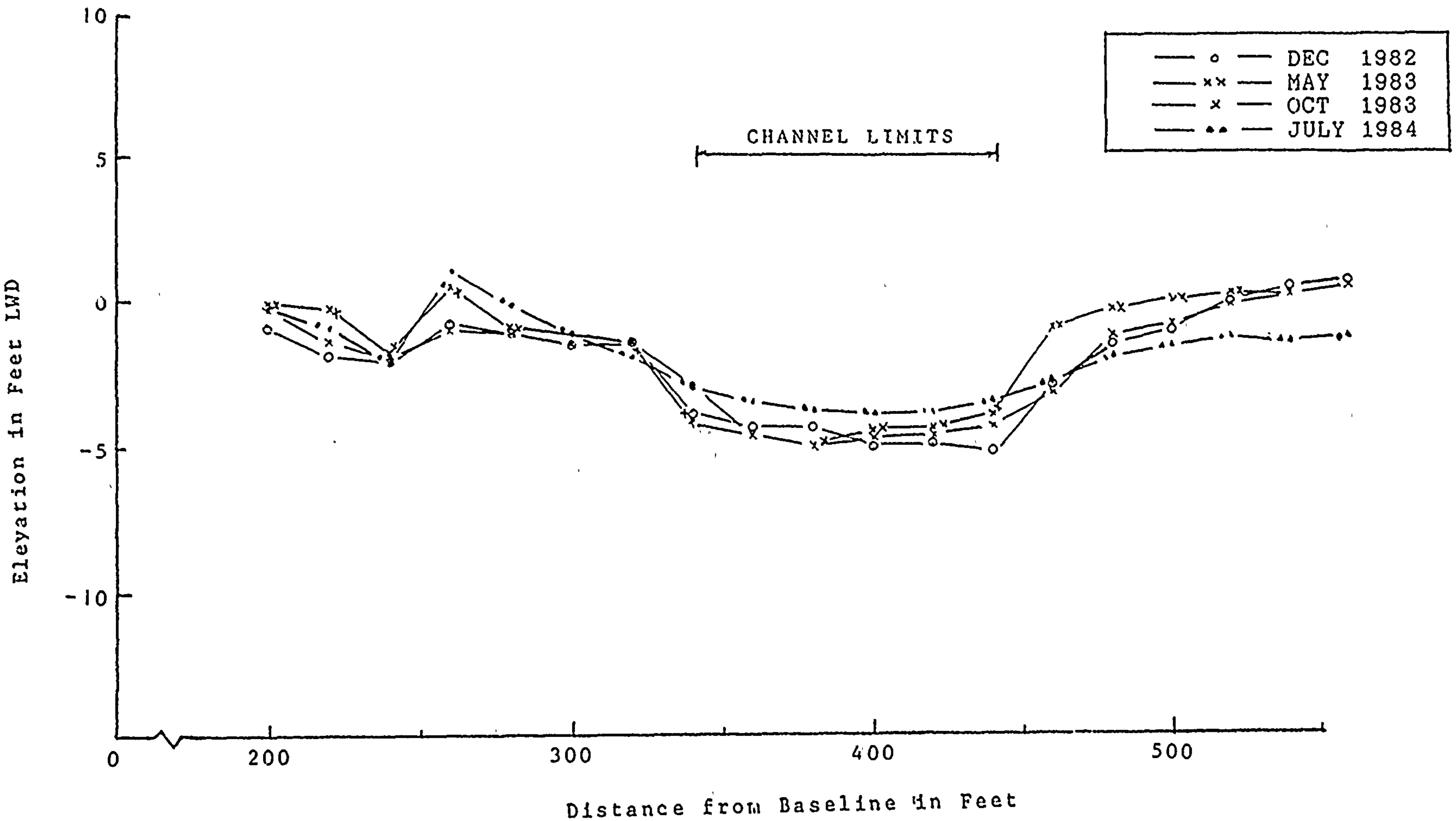


Figure 82. Channel cross sections for Sta 80+00, 1982-1984

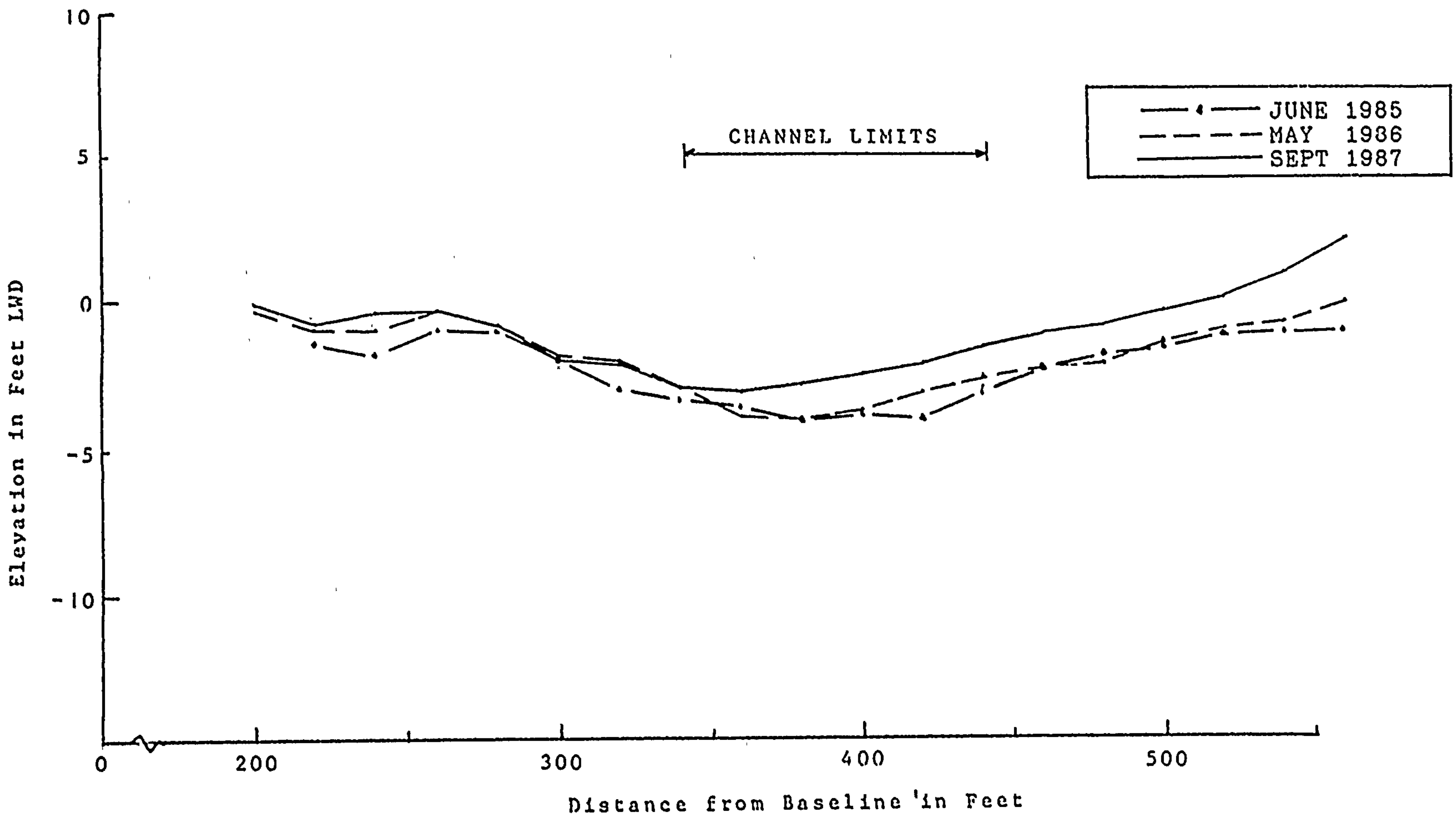


Figure 83. Channel cross sections for Sta 80+00, 1985-1987

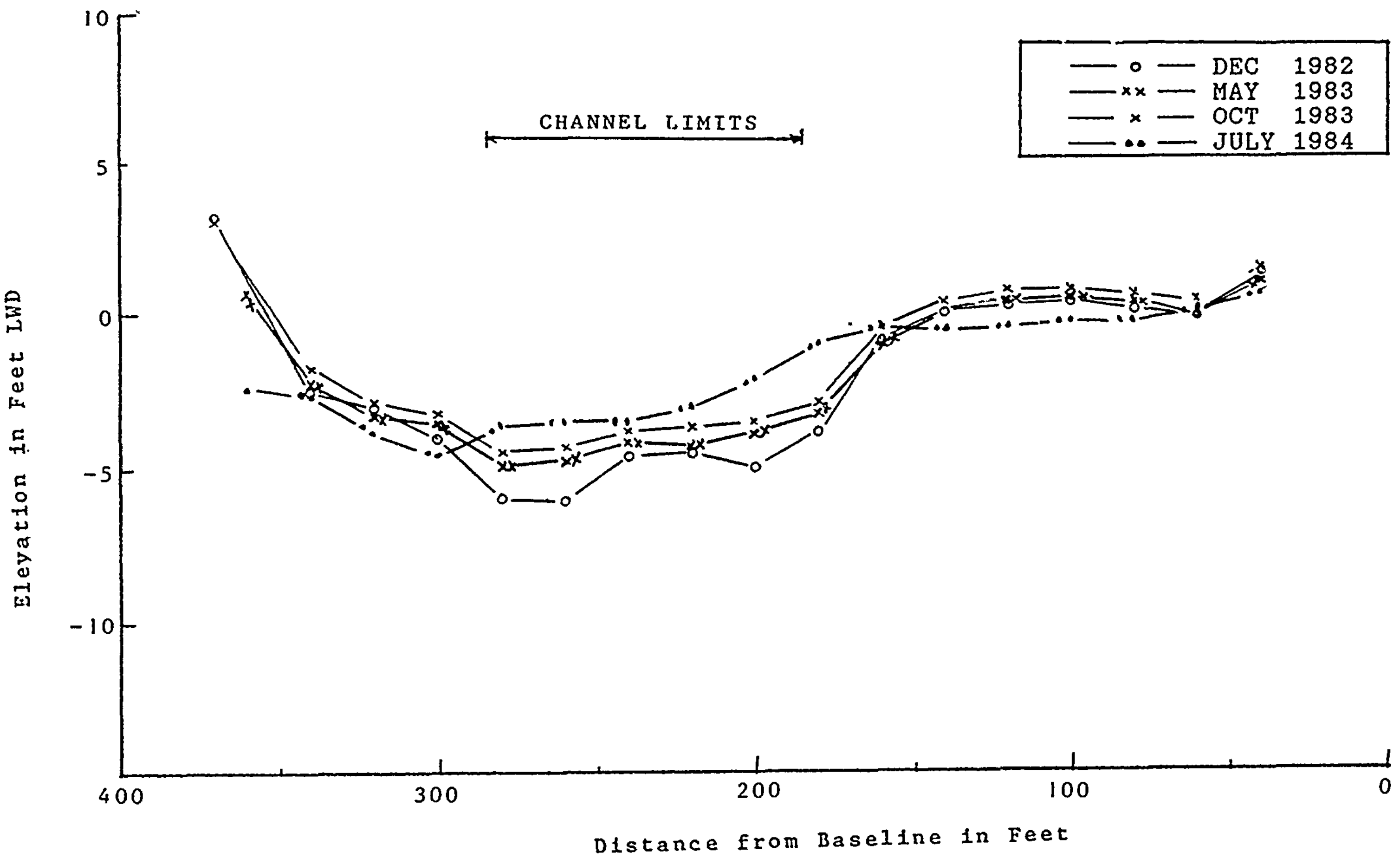


Figure 84. Channel cross sections for Sta 90+00, 1982-1984

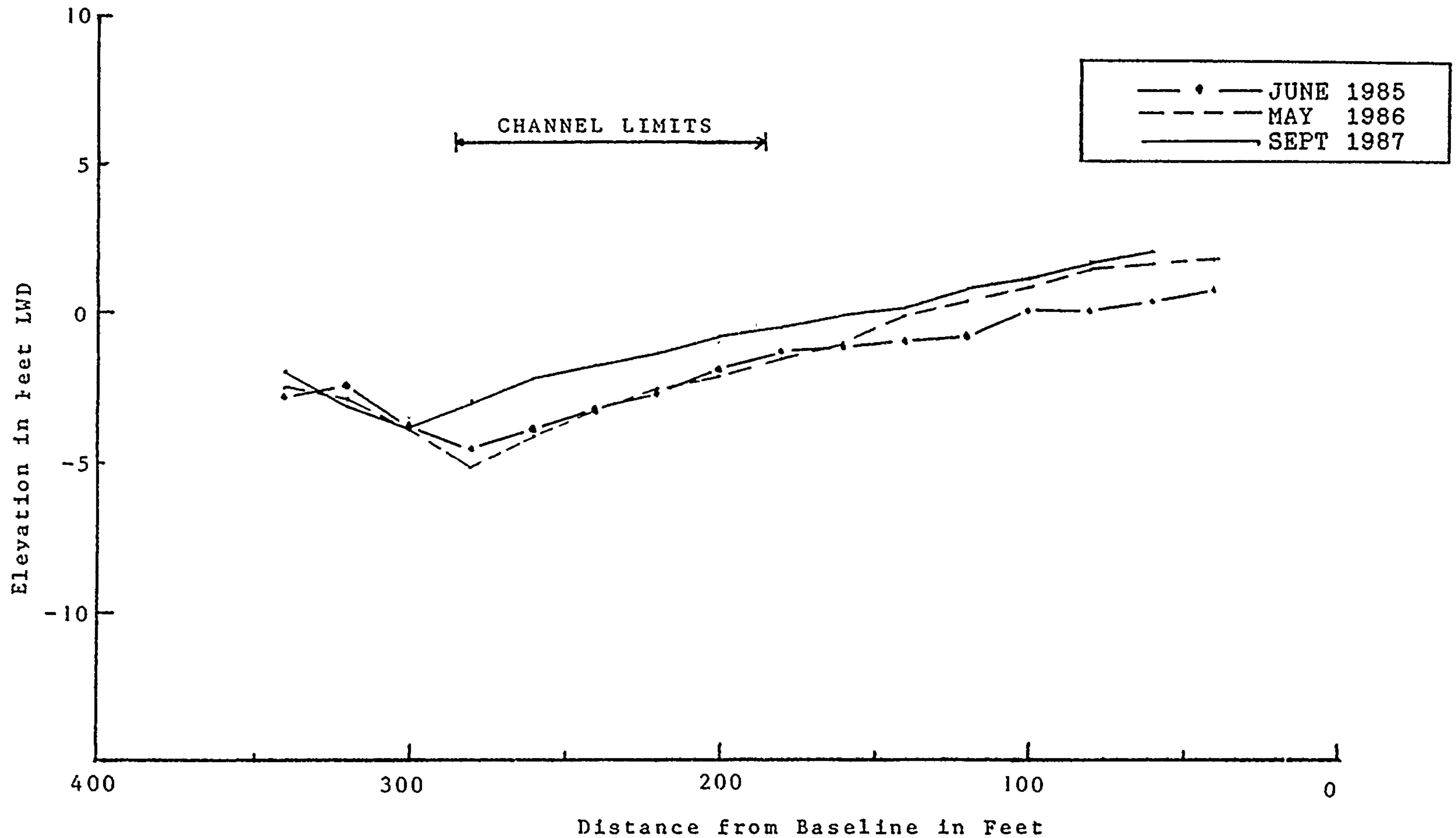


Figure 85. Channel cross sections for Sta 90+00, 1985-1987

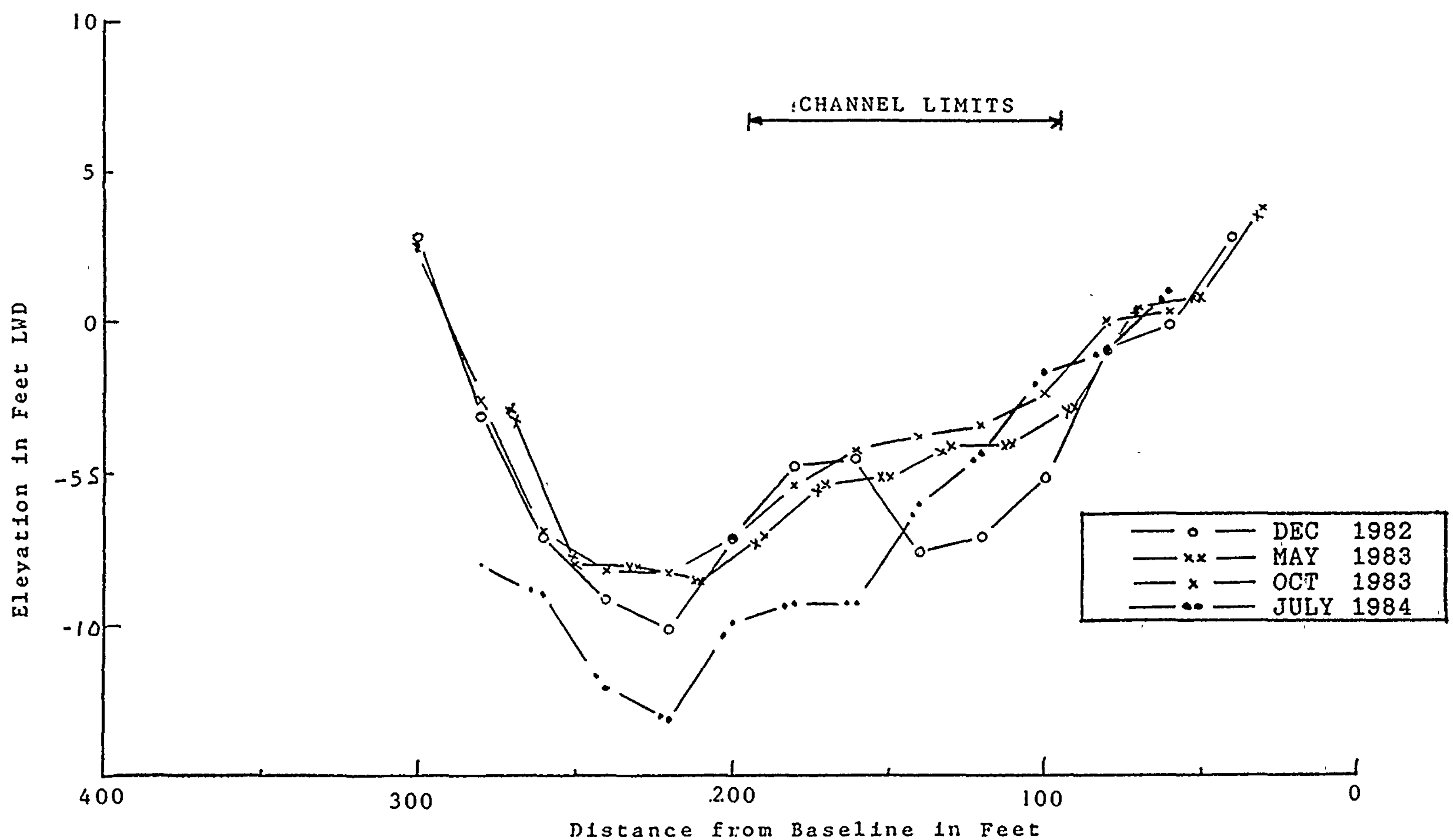


Figure 86. Channel cross sections for Sta 100+00, 1982-1984

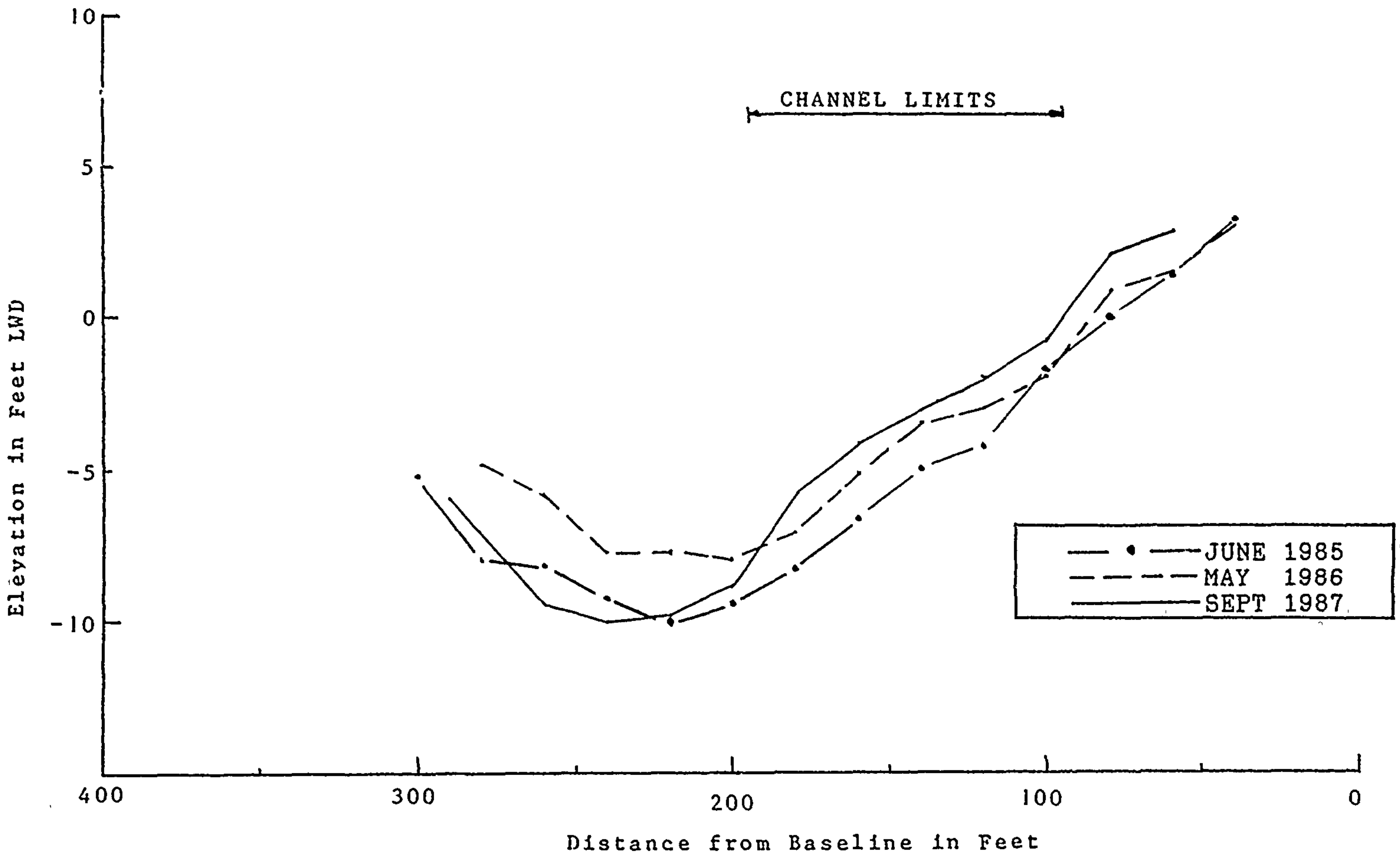


Figure 87. Channel cross sections for Sta 100+00, 1985-1987

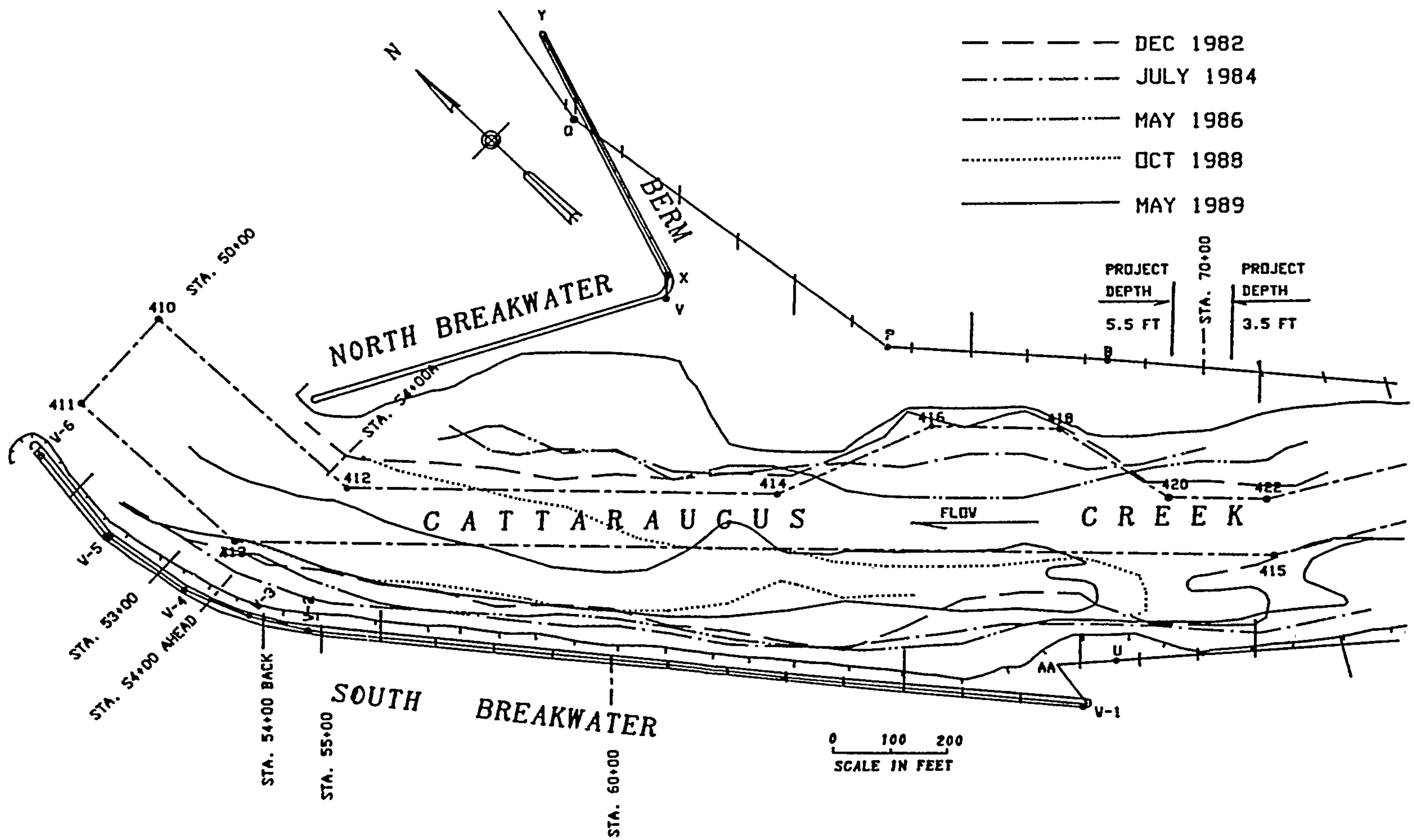


Figure 88. Shoaling and movement of -3.5 ft LWD contour

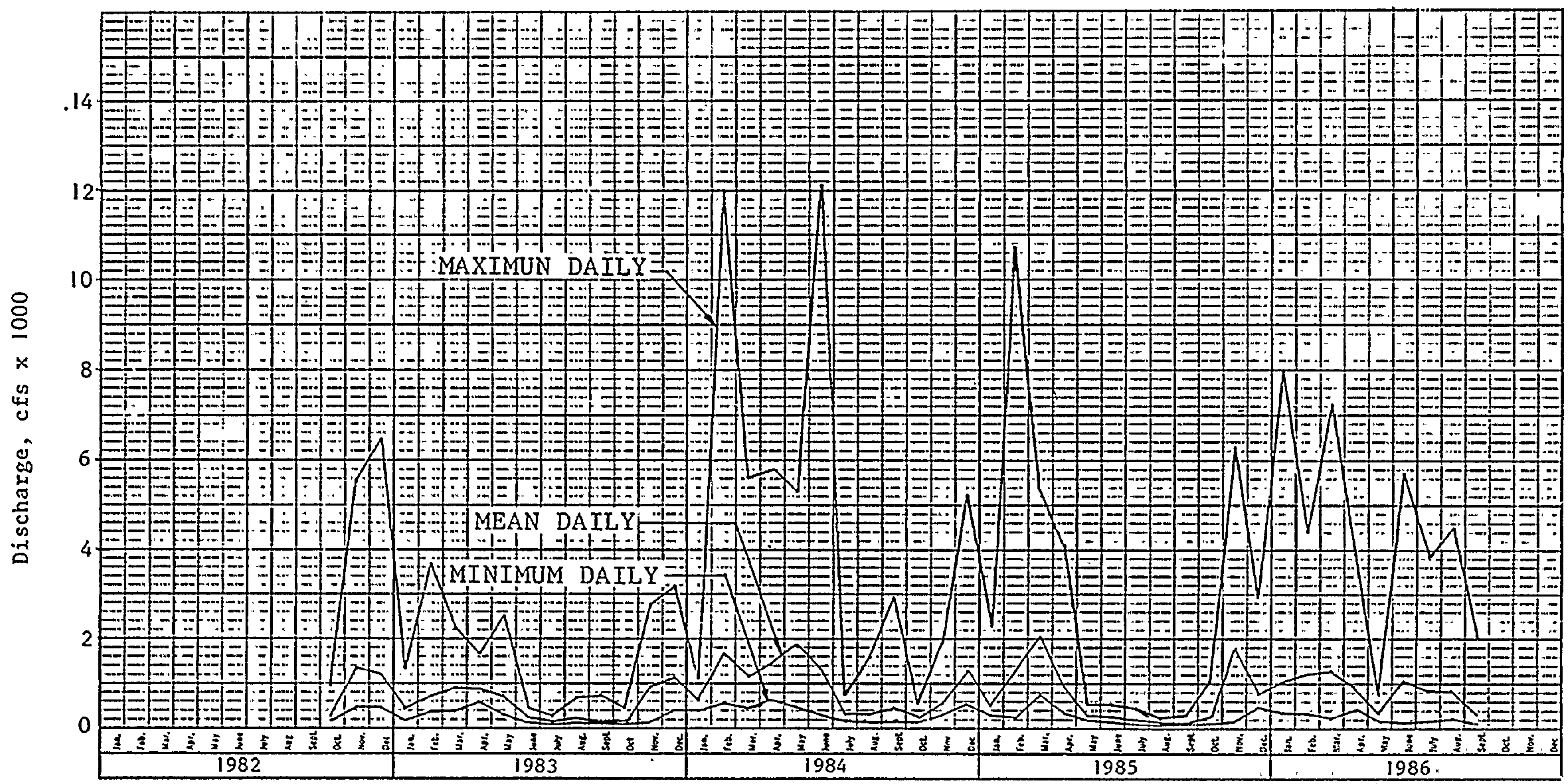


Figure 89. Cattaraugus Creek discharge at Gowanda; minimum, mean, and maximum daily discharge

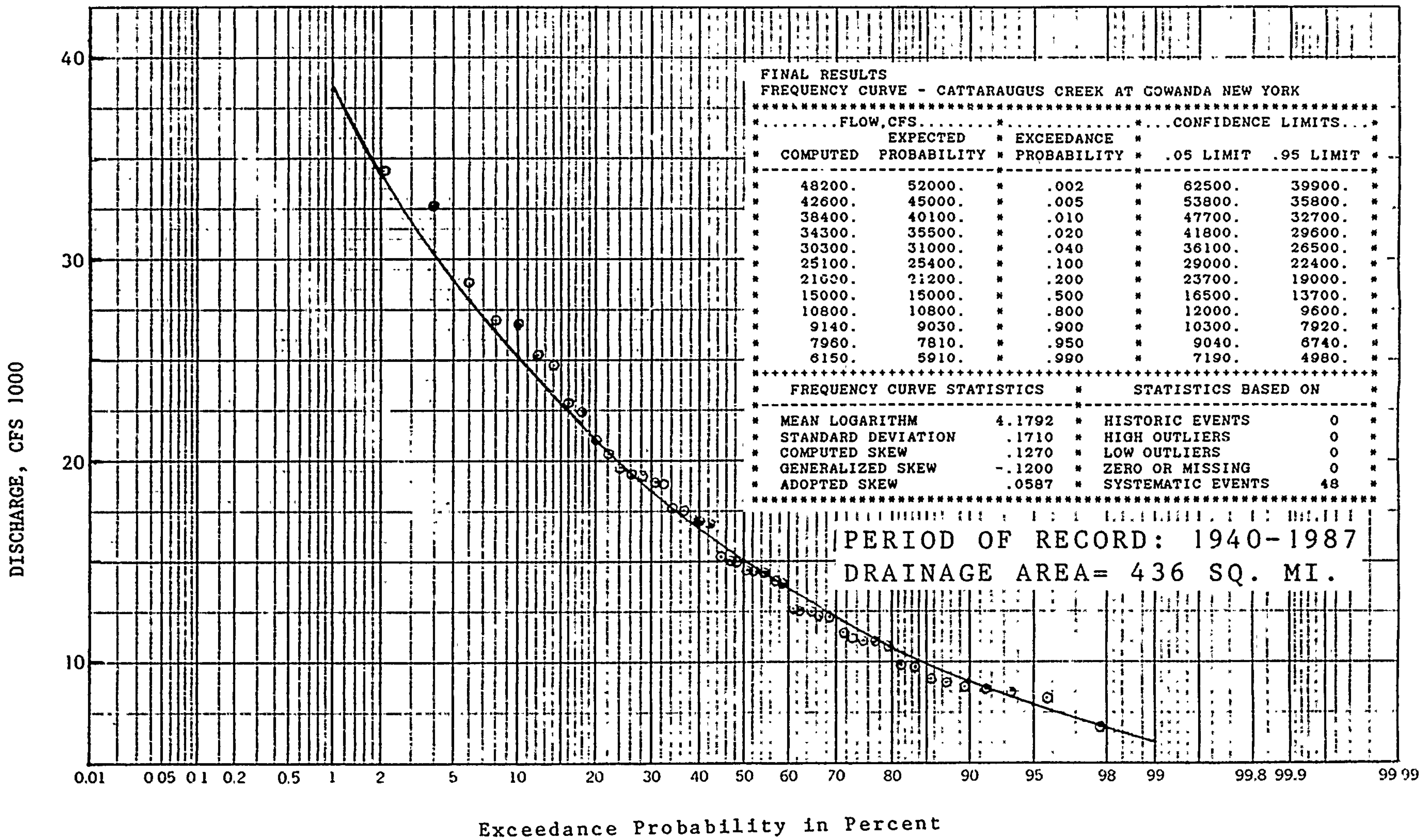


Figure 90. Discharge-frequency curve for Cattaraugus Creek at Gowanda

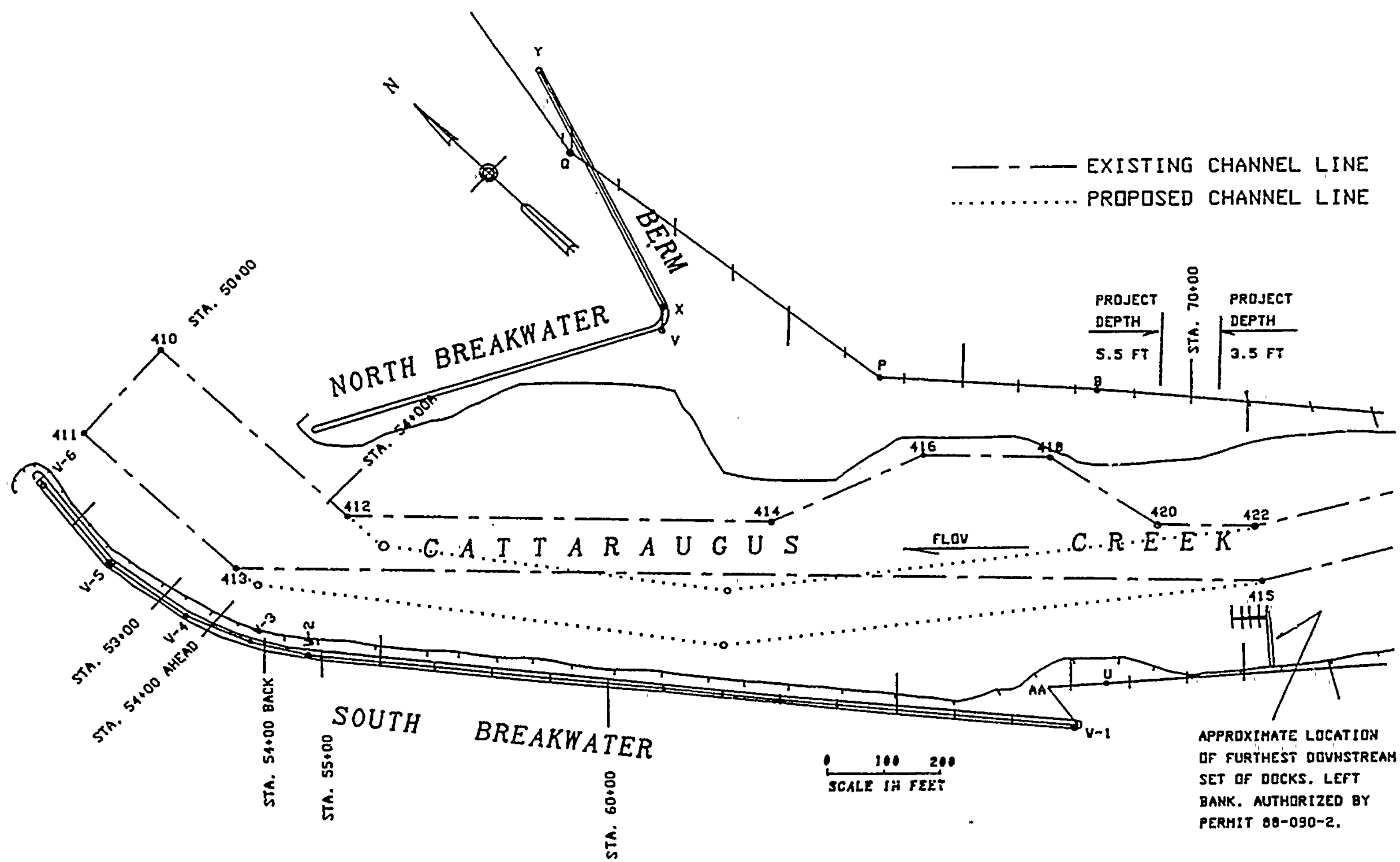


Figure 91. First proposed downstream channel line

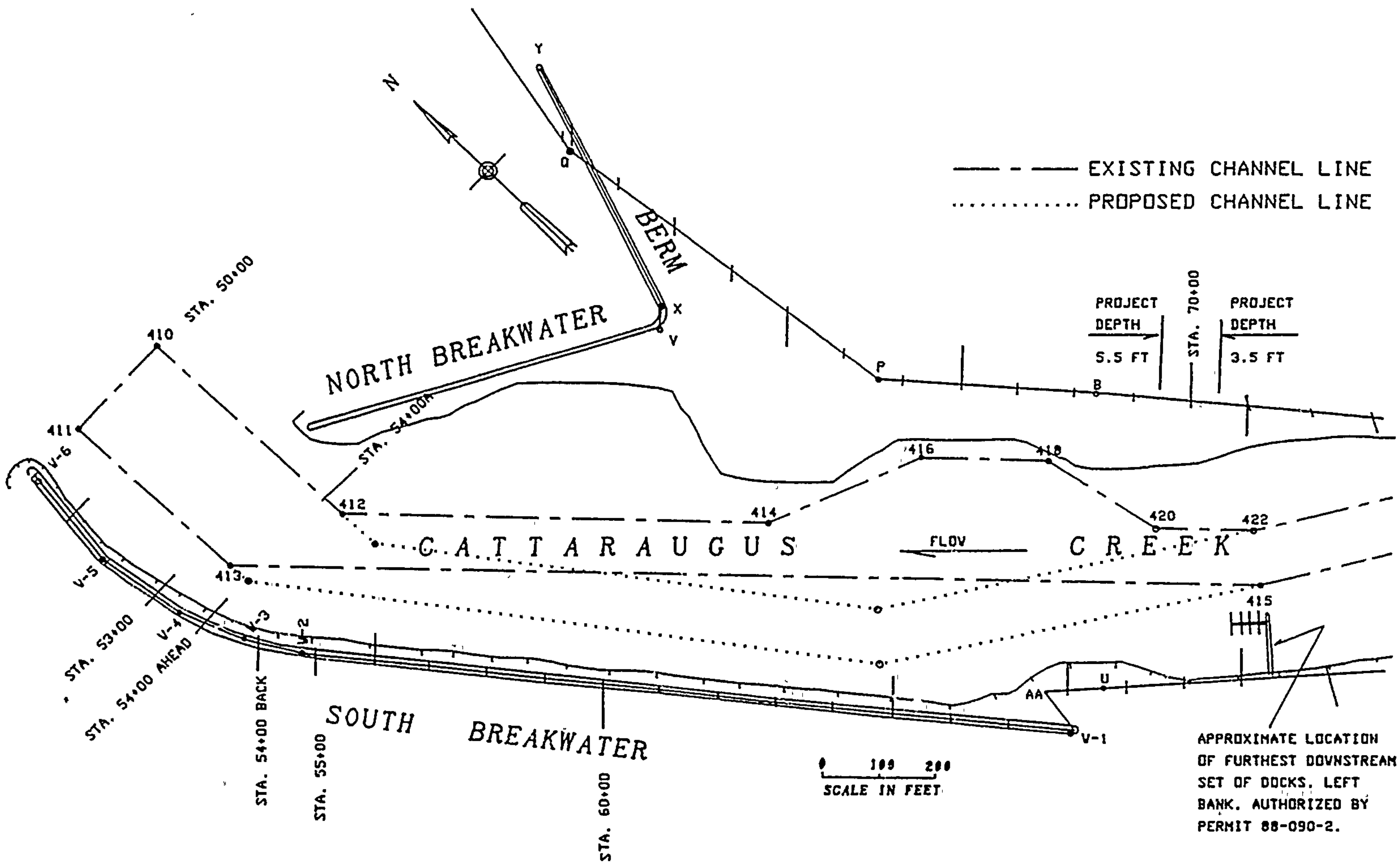


Figure 92. Second proposed downstream channel line

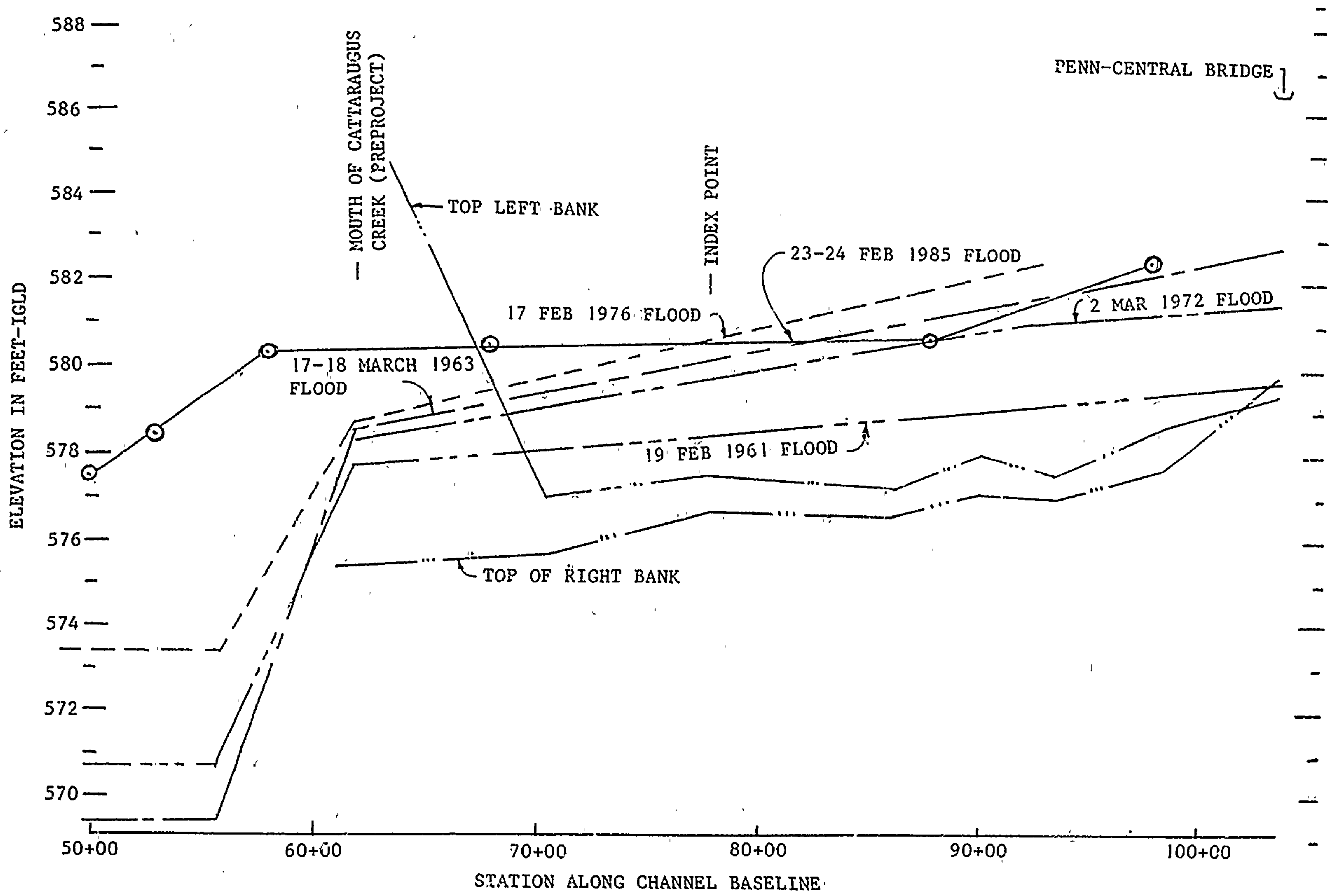


Figure 93. Flood profiles

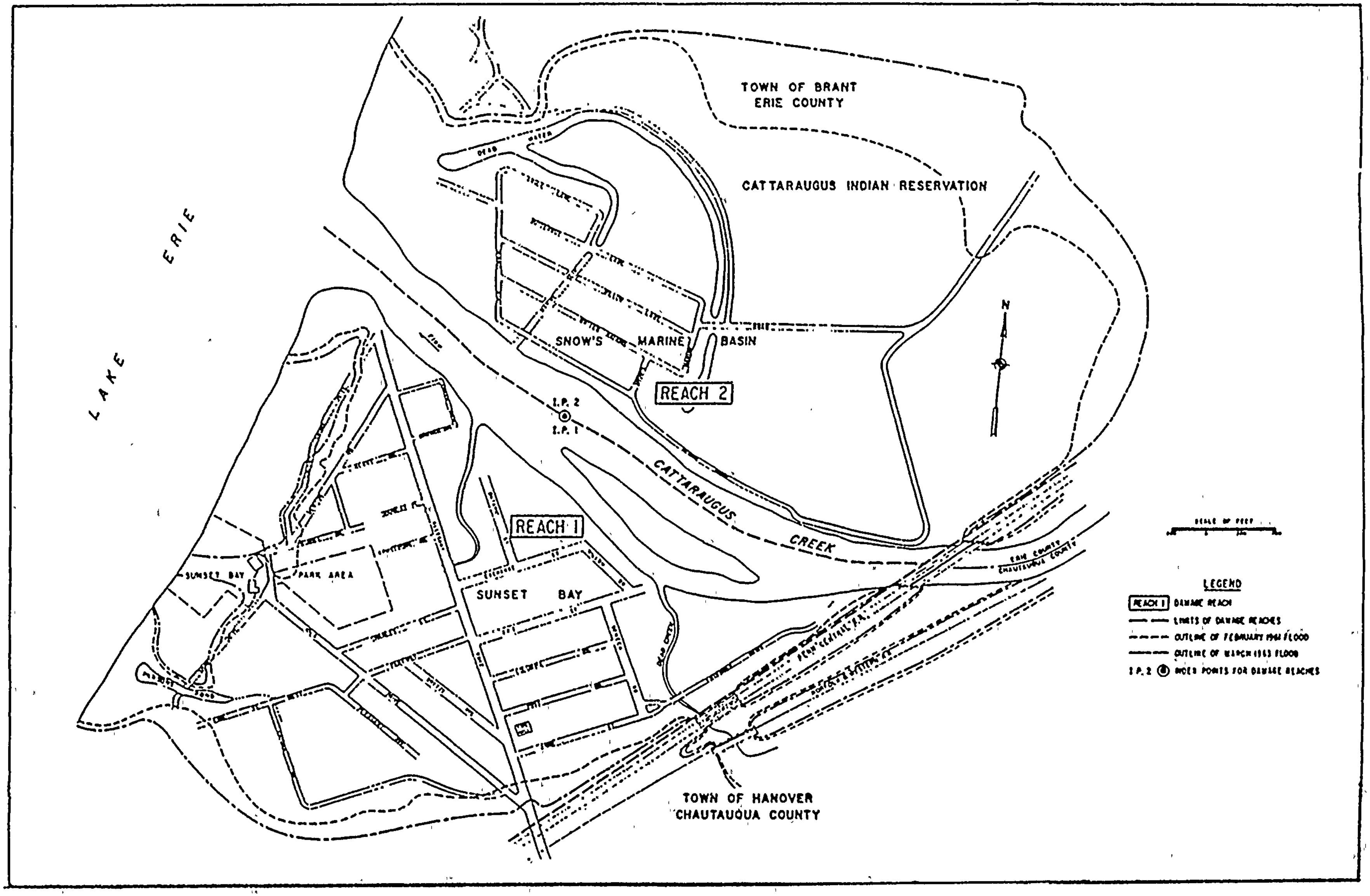


Figure 94. Flooded outlines for the 1961 and 1963 ice jam

Figure 95. Right overbank near the mouth, 25 Feb 1985

Figure 96. Cattaraugus Creek mouth, 25 February 1985

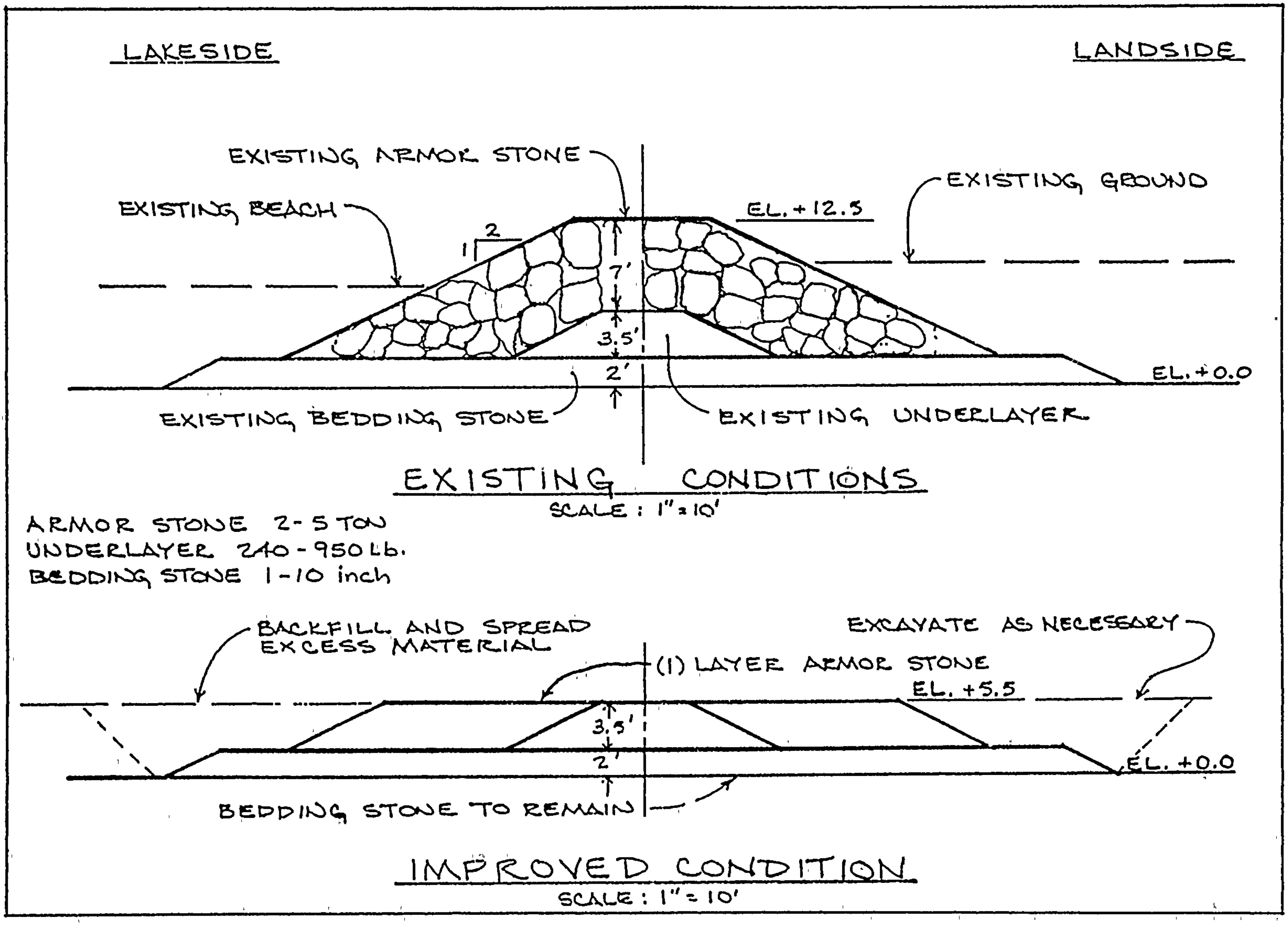


Figure 97. Sketch of cross section for berm height reduction

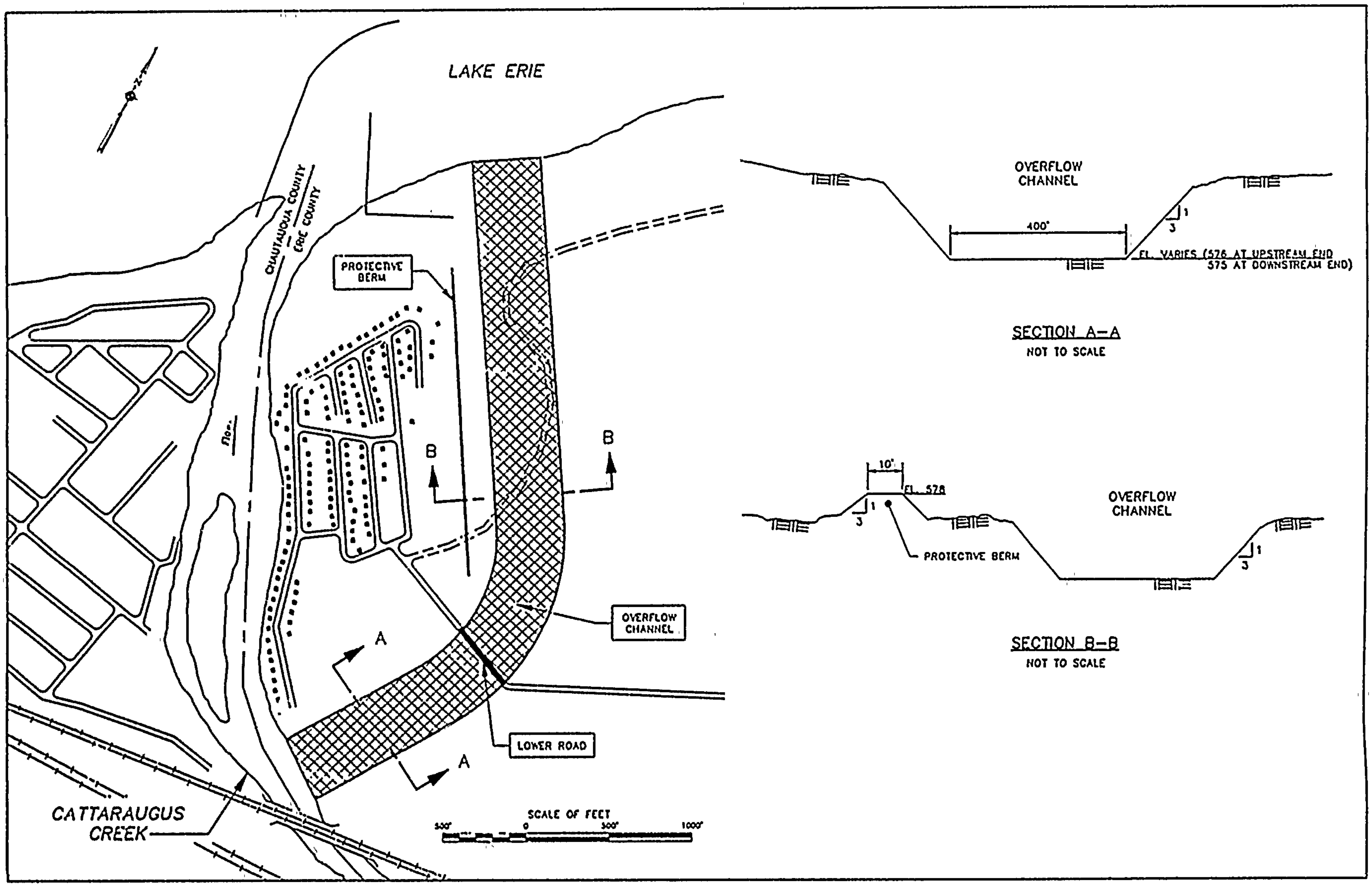


Figure 98. Alternative Plan 3A

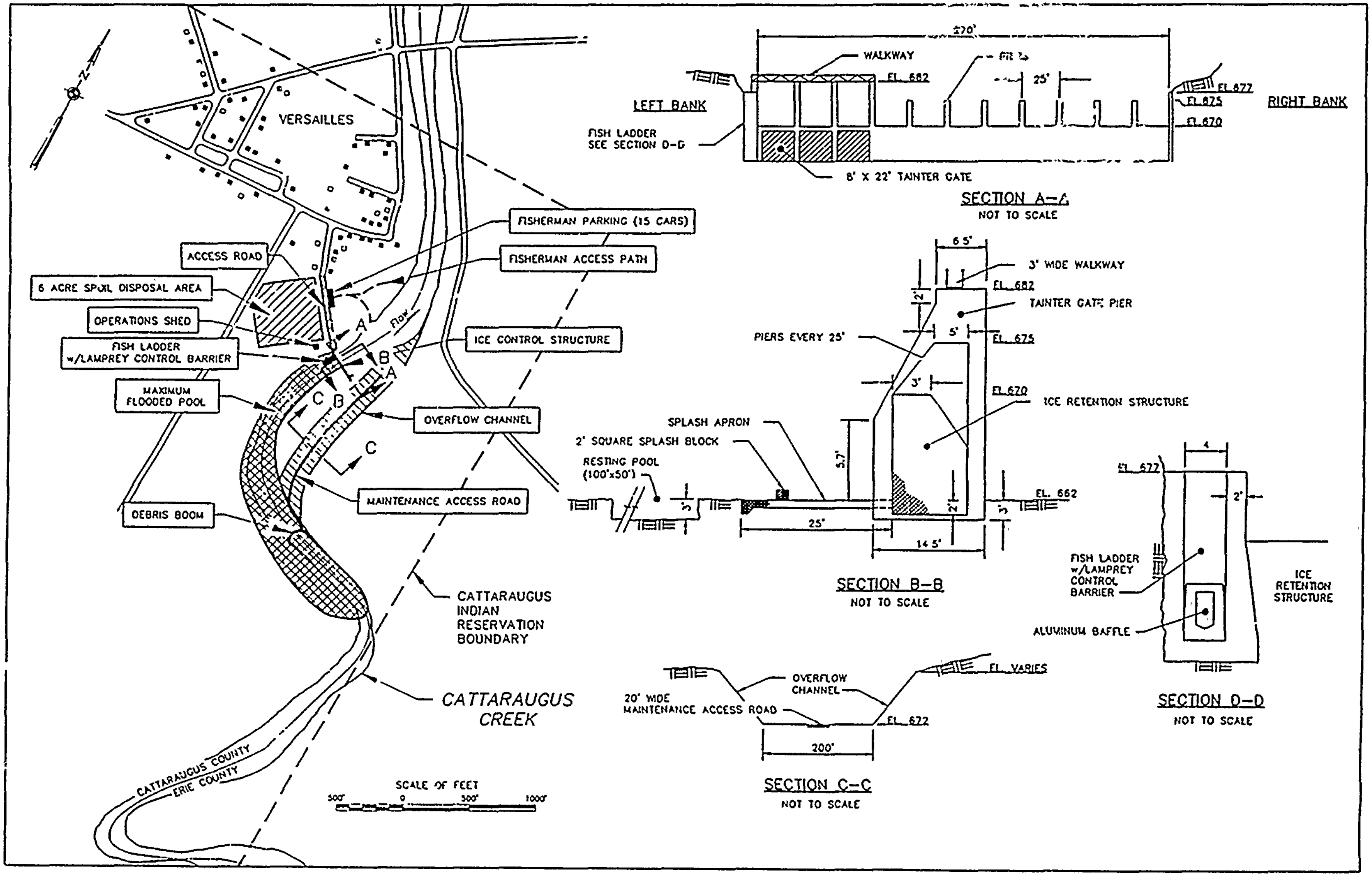


Figure 99. Tentatively selected Plan 3B(2), modified

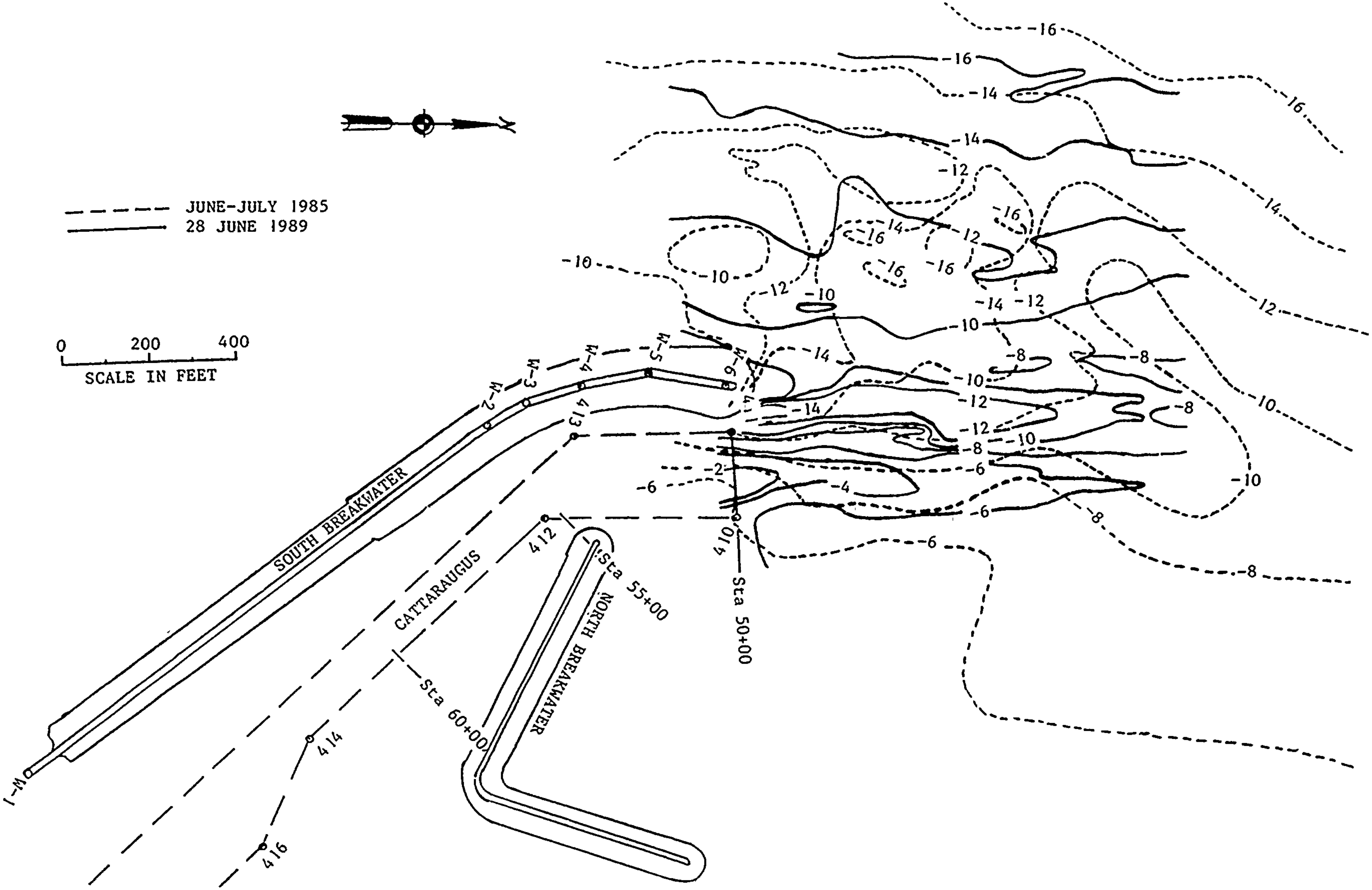


Figure 100. Offshore bathymetry changes in the vicinity of the south breakwater head, 1985-1989

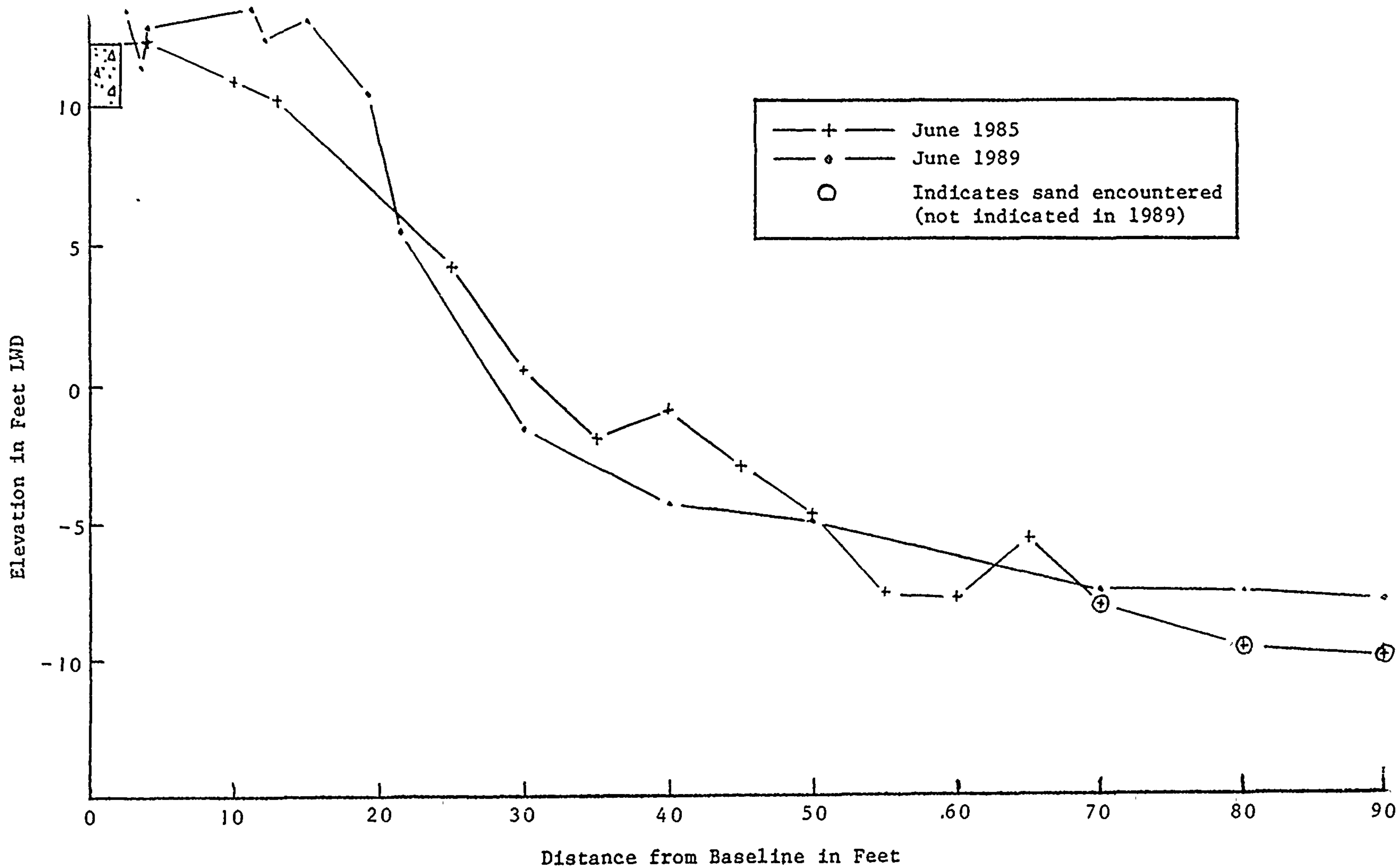


Figure 101. Structure cross section for W-6 (W)

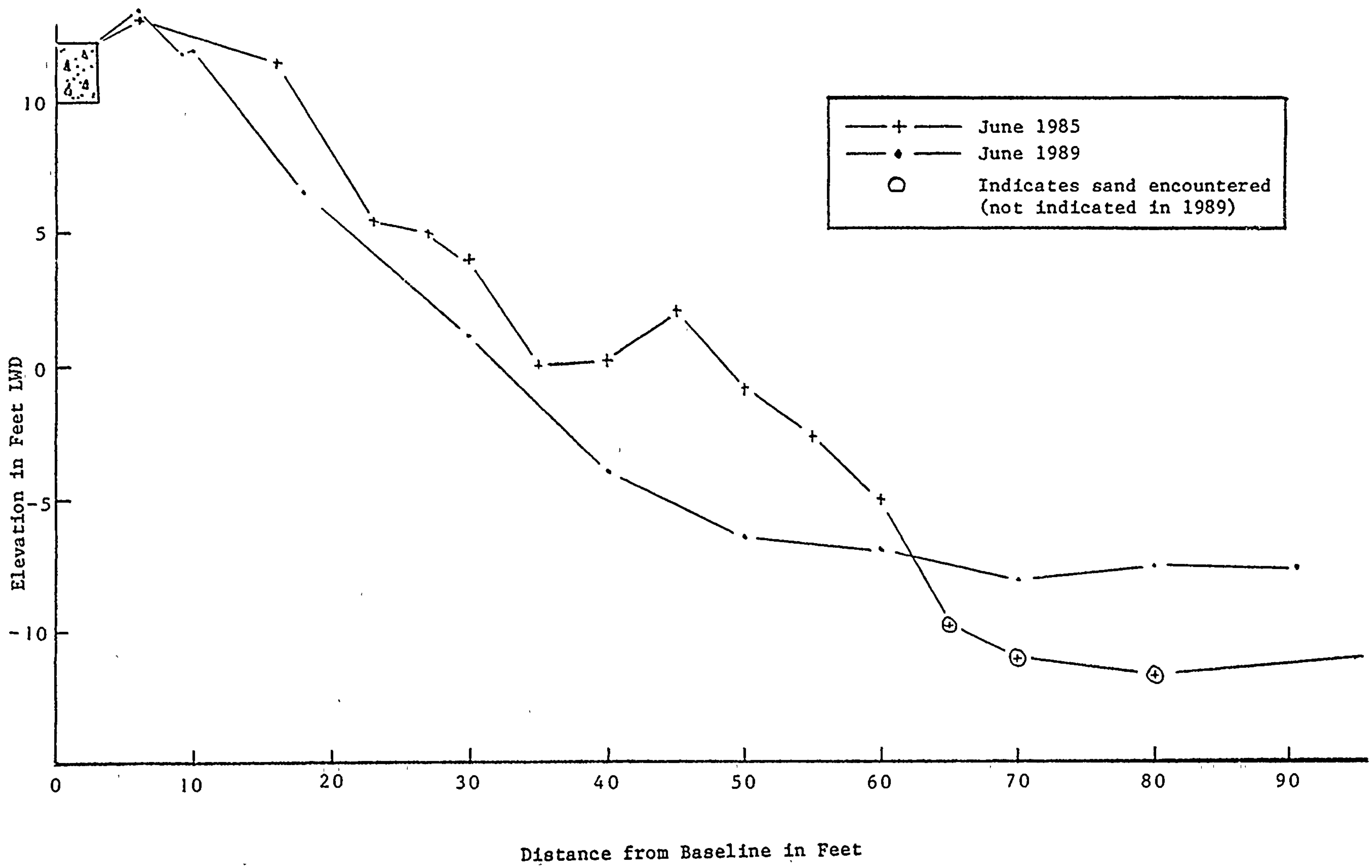


Figure 102. Structure cross section for W-6 (NW)

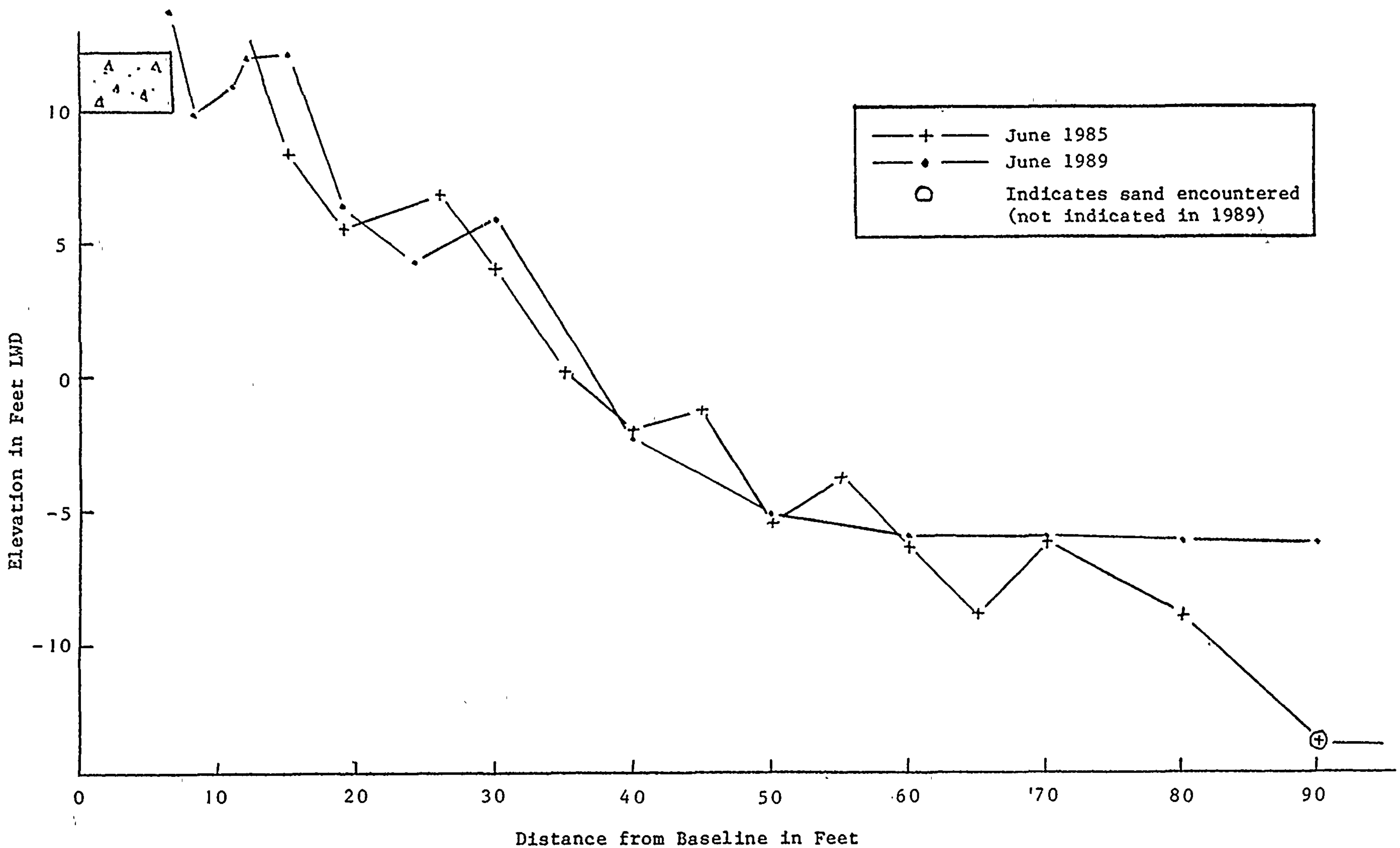


Figure 103. Structure cross section for W-6 (N)

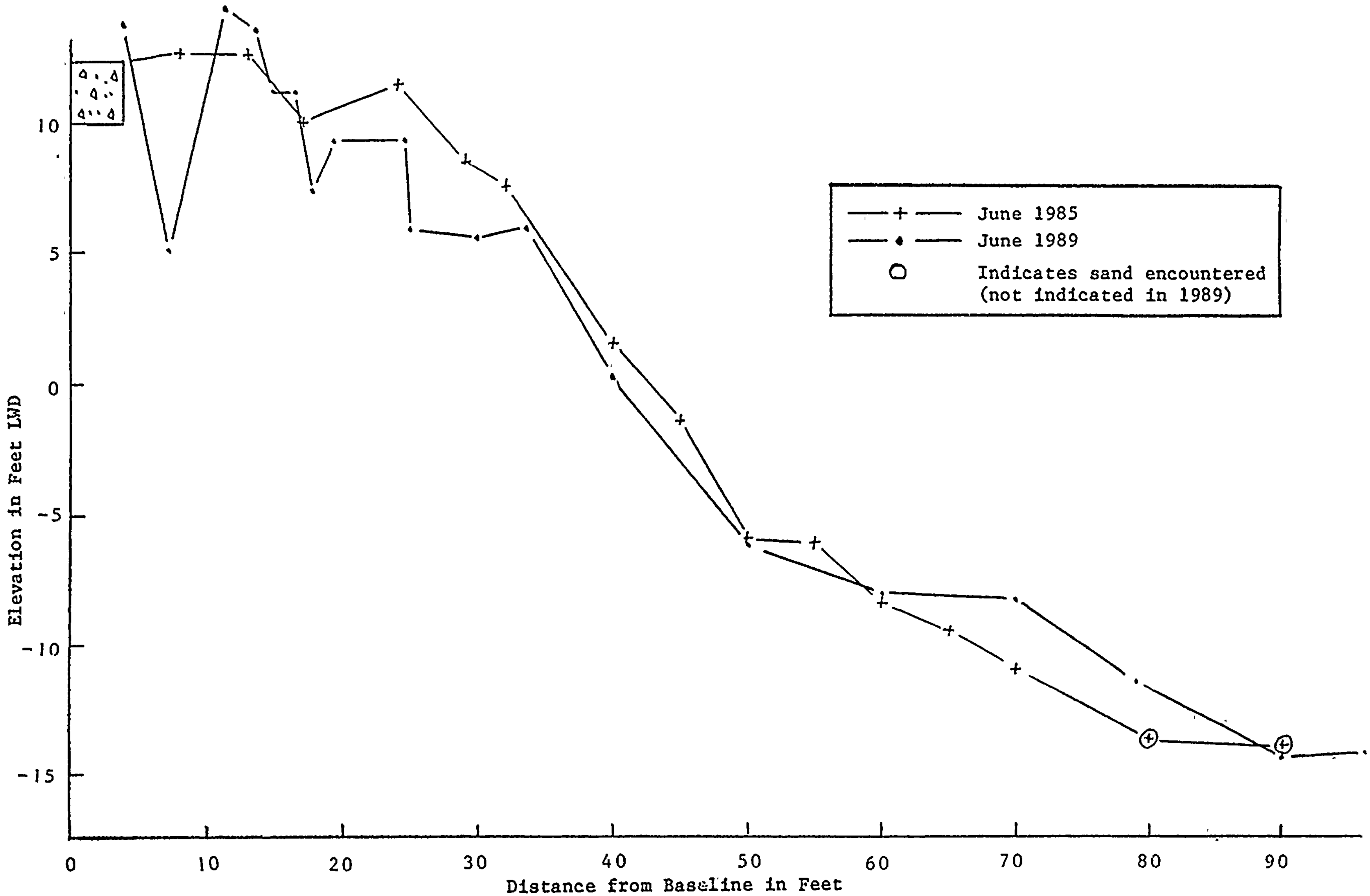


Figure 104. Structure cross section for W-6 (NE)

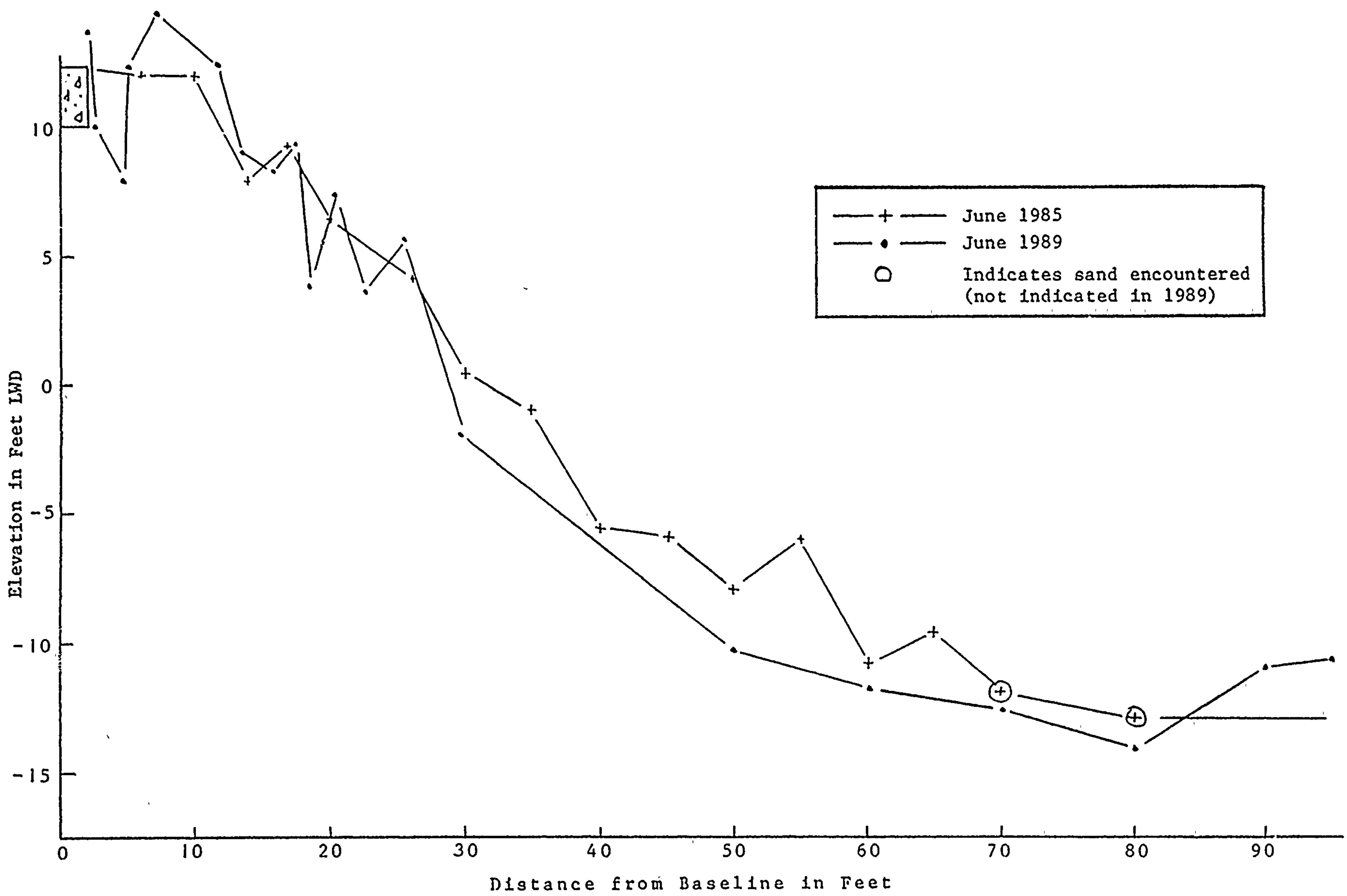


Figure 105. Structure cross section for W-6 (E)

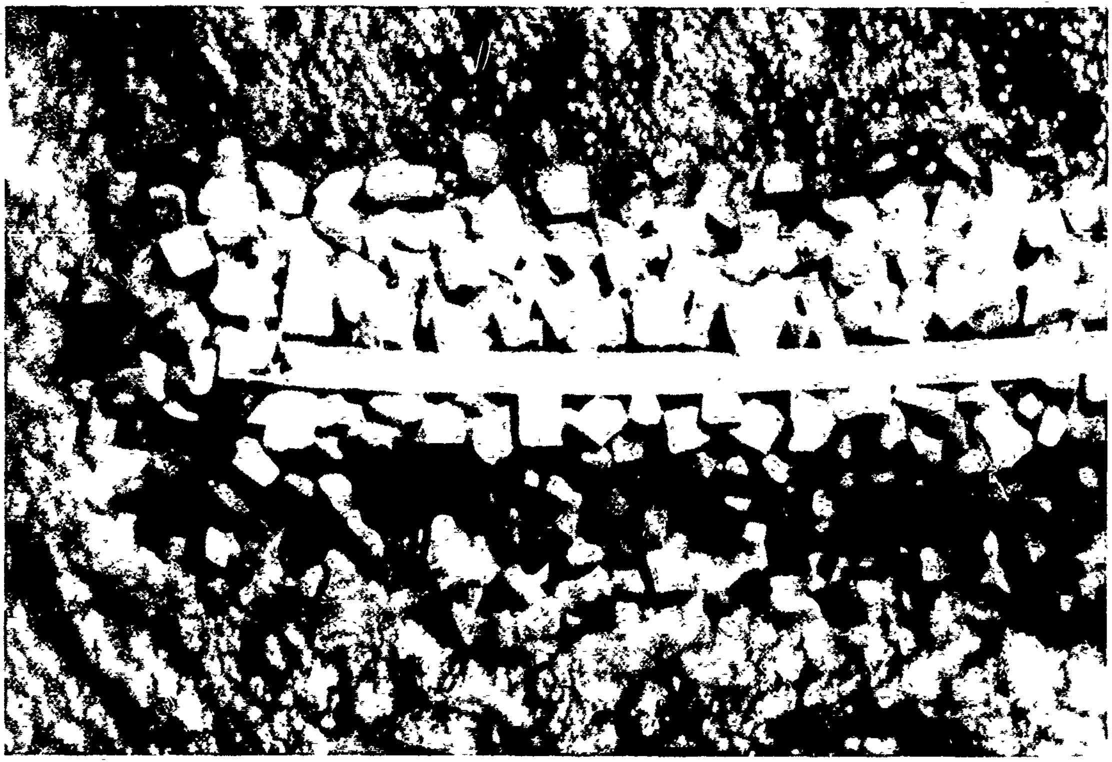


Figure 106. South breakwater head, 16 May 1984

Figure 107. South breakwater head, 3 June 1986